Water-Supply Engineering

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The Designing and Constructing of Water-Supply Systems

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WHOSE INTELLIGENT AND PAINSTAKING CRITICISM
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IN THE REWRITING OF THIS
NEW EDITION

PREFACE TO THE THIRD EDITION

DURING the eighteen years since the first edition of this book was written there have been great advancements in water works theory and practice, especially in those branches having to do with the quality of water. Consequently, not merely a revising but an entire rewriting of the book seemed to be necessary, if it was to continue to be of any service to water works students and practitioners.

In the present edition the matter treating of water purification and the quality of water has been entirely rewritten at much greater length; that describing pumping machinery also is practically new throughout; and hardly a paragraph of the original text has been used without some change calculated to bring the subject up to date. This resulted in considerably increasing the length of a work that was already too long, from some points of view; and Part II of the first two editions, dealing with Construction, has been condensed to a single chapter, and Part III, dealing with Maintenance, has been omitted altogether. Also the chapter on Hydraulics has been omitted, since those using the book are assumed to be familiar with and to possess a text book discussing the principles of this subject. Several of the chapters have been rearranged so as to present the subject matter in a somewhat more logical order.

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CHAPTER I

SYNOPSIS

ARTICLE 1. SOURCES OF SUPPLY

ALL mankind, whether wandering savage, isolated pioneer, or dweller in a crowded city, must have water for drinking, and needs it for cleansing himself and his belongings; and under many circumstances he uses it for irrigation, for power, or in manufacturing, as well as for extinguishing fires, sprinkling lawns and streets, flushing sewers, and many other purposes. The water for all these uses can have but one first source—the moisture in the atmosphere. This generally becomes available as rain or snow, but dew is in some cases an important consideration.

Water falling as rain or snow may be caught before reaching the ground and stored in basins or cisterns hewn from the rock, dug in the soil, or in the shape of tanks of wood, metal or masonry. Or it may be taken from rivers or smaller streams direct, or from natural reservoirs, i.e., lakes; or the streams may be intercepted and stored in large artificial reservoirs. Or that which soaks into the ground may be obtained by wells, shallow or deep, dug or driven; or at its emergence from the ground in the form of springs.

ART. 2. REQUISITES OF A SUPPLY

By a water supply is meant the water that is supplied or brought, generally by artificial means, to the point or points where its use is desired. It may be supplied at a power house

for operating hydraulic machinery, at a farm for irrigation, or at hundreds and thousands of fixtures in a city for household or manufacturing purposes, flushing streets and sewers, extinguishing fires and other purposes connected with the complex life of the citizens.

Each of these purposes has its individual requisites. For flushing streets and sewers and extinguishing fires, the chief ones are volume and ready accessibility, although considerable pressure at the outlet is generally desirable for the last. However, any considerable amount of suspended matter (clay, leaves, etc.) is objectionable. For manufacturing purposes a clear water is necessary in many cases, and in some the presence of certain matters in solution may be highly objectionable, while on the other hand that of certain others may be desirable. For general household purposes a clear and fairly pure water is very desirable, and for drinking and use in connection with food the water should be entirely free from matters which may cause sickness or physical disorders of any kind. Each of these uses also has its special needs as to quantity.

Aside from quality and quantity, the principal requisite of a supply is convenience of the consumers in obtaining and using it. In many European cities and a few American ones there are neighborhood conveniences—fountains, pumps, wells, etc.—from which the residents in the vicinity obtain their supply in pails or other receptacles; but modern civilization looks upon at least one outlet in each house as a necessity, and most houses have many such. For fire extinguishing, fire hydrants at intervals of a few hundred feet are demanded, although cisterns from which water can be drawn are still used to some extent. And for other uses, special forms of outlet are provided. In cities the water must be forced to and out of these various outlets, even when they are on elevated land and the top floors of buildings.

We have, therefore, as the chief requisites quality, quantity, and convenience for utilization.

Art. 3. Elements of a System

Before considering in detail the designing of a system for supplying water, it will be well to understand what elements go to make up the complex whole, in order that the interrelation of each with all the others may be understood. These elements may be classified under the following heads:

(1) Head works. Those necessary for bringing the supply under control and directing it into suitable conduits. These include dams, head-gates, impounding reservoirs, and the tributary catchment areas; wells; intakes in streams or lakes; pumps.

(2) Purification plants, for improving the quality of the water where this is necessary. (These might be included under head works, but are of such importance as to warrant a separate classification.)

(3) Regulating reservoirs. Reservoirs and standpipes for controlling the pressure and rate of supply.

(4) Distribution system. Conduits for conducting the supply to the points of service, both open conduits and pipe lines; also service appurtenances, such as faucets, fire hydrants, meters, penstocks for power plants, and other appliances and features for special uses.

Each of these will vary in magnitude and complexity with the volume of water dealt with and other conditions of the problem.

Head Works. The water falling as rain is received on a catchment area (sometimes called watershed) where it flows by gravity to streams, or soaks into the ground to reappear as springs or be drawn from it by wells. The catchment area for a single family collecting its own supply may be the roof of the house; for a large city it may contain hundreds of acres, which drain to streams whose flow is caught and retained in one or more impounding reservoirs. Or, if the supply is drawn from a large river or lake, the catchment area includes the entire section that drains to the river or lake above the point of intake. In the case of a well, the catchment area is the country, some-

times hundreds of miles in extent, which supplies, by the seepage into it, the underground flow which the well taps.

Where the water is impounded, a reservoir for this purpose must be constructed, and provision made for withdrawing the water into the distribution system. If the water is from a lake or deep river, an intake pipe leads it to the distribution system or to a pumping plant, this pipe terminating at its outer end in an intake, a more or less elaborate structure for controlling the entering of the water into the intake pipe. If the water is drawn from a well, it may be necessary to pump the water to the surface of the ground, or it may flow of itself into a surface pipe; which pipe connects with the pumping plant when pumping is necessary, as it almost always is. In case the supply is obtained from a spring, this is generally enlarged into a basin or small reservoir, from which it is piped. Underground water may be intercepted in other ways also, which will be described in the appropriate place.

Where the impounding reservoir, stream, lake, well, or other point where the water is first brought under control and intercepted is not sufficiently high to force the water by gravity to the point of utilization and furnish sufficient pressure there, pumping must be employed.

Purification Plants. If the water is not suitable in quality for the purposes for which it is to be used, it should be made so by proper treatment, physical, chemical, and bacteriological. This treatment is not strictly "purification," but is called so for convenience; it is generally a removal of a part or all of certain of the matters carried in suspension or solution, but is often a mere modification of contained matters into unobjectionable forms or conditions. The plants for so treating the water are generally placed adjacent to the head works, but may be anywhere between these and the point where the supply is first utilized.

Regulating Reservoirs. The head works may be located at such an elevation above the city or other point of utilization as to give an undesirable pressure in the pipes at such point; in which case the water may be carried from the head works

to a distributing reservoir or standpipe so situated as to give the pressure desired. Or the water may be pumped to the distribution system; in which case a reservoir or standpipe is desirable to maintain the pressure constant, to store water for emergency use (such as fires) at a rate greater than the pumps can supply it, and to maintain the supply while the pumps are idle.

If the flow of the stream that furnishes the supply falls below the rate of consumption at any time, there should be a supplementary supply to make up the deficiency. This is best provided by storing up the surplus flow at the times when the stream supplies more than is being consumed. Reservoirs for so equalizing the supply are called storage reservoirs. Every impounding reservoir is more or less of a storage reservoir also; but in many cases special storage reservoirs must be provided for this purpose alone.

There are also numerous other purposes connected with regulating the supply for which reservoirs and standpipes are used.

Distribution System. This generally consists of pipes; although for irrigation, canals and ditches comprise a large part; and canals, flumes, tunnels and other forms of conduit are used in many cases between head works and distributing reservoir or as parts of the head works. These must be designed of the size which will carry the amount of water required, and of a strength to resist the pressure of the contained water, without an unnecessary sacrifice of economy.

In general, all of the elements of a water-works system must be designed to harmonize and supplement each other most effectively. In some cases one feature may do duty in several ways; an impounding reservoir, for instance, may serve also not only as a storage reservoir, but as an element of the purification plant—a sedimentation basin—as well. All of the elements should work cooperatively toward the one end—the supplying of water of a desirable quality, in the desired quantities, in a way and under conditions making for convenience and safety of the consumers.

CHAPTER II

REQUISITES OF A SUPPLY. QUANTITY

Art. 4. Total Annual Rate

The designing of most of the individual features of a waterworks system—pumps, reservoir, mains, etc.—is based upon some quantity of water assumed or estimated as that required to meet the needs of the city. But different quantities are used in the calculations of the several features, for reasons to be explained.

Every part of a permanent plant must be designed to meet the needs of some future time, and since the city and its demands for water service are constantly growing, the future needs will be greater than the present. How distant a future should be provided for must be determined separately for each part of the plant. In general, it may be said that it is wasteful to provide a capacity that will not be reached before the structure or appliance in question has been worn out or before the time arrives when it should be replaced by some more efficient or effective substitute. Also, it will not generally be economical to provide a pump or other machine of a capacity so beyond its present requirements that its present operation is uneconomical. On the other hand, it is uneconomical to provide a pump, pipe line, etc., so small that it must be duplicated in too short time. The determination of the length of time to be provided for will be discussed in a following article.

The quantity will be expressed in gallons per minute, hour, day, week, year, or number of years, according to the use to be made of the quantity. The rate per minute at which water is used at a fire is not often continued for more than an hour or two (ten hours is considered the maximum time by the National Board of Fire Underwriters); and differs enormously from that of domestic consumption calculated by the day.

And the maximum daily consumption is much greater than the average for a year. But each of these and other rates are used in the different calculations in the preparation of the plan.

A public supply is used for a great variety of purposes. The amount of water actually drunk seldom exceeds 1 per cent of the consumption. About 25 to 50 per cent is ordinarily used for household purposes (including lawn sprinkling, washing, etc.). This is called domestic use. The non-domestic uses are classified by the American Water Works Association as Industrial (factories, railroads, etc.), Commercial (stores, saloons, stables, hotels, etc.), and Public (fire protection, street sprinkling, sewer flushing, public buildings, schools, fountains, etc.). In addition, there is an amount that is Unaccounted For, largely leakage from pipes and fixtures or other parts of the system.

Domestic use is often assumed to vary directly as the population, but this is seldom the case, especially with small cities. For as a city grows, a larger percentage of the population uses the public supply, and each consumer is more lavish in his use, partly because he introduces more plumbing fixtures into his house. This is illustrated in Table No. 1, prepared from figures collected by the University of Iowa, in which are given data concerning 30 Iowa cities, arranged in the order of size of population. Of the ten largest, 72.6 per cent of the population used the public supply, while of the ten smallest only 53.2 per cent used it. Domestic consumption may, however, be assumed to be a *function* of population only, but increasing by a power somewhat greater than the first.

Domestic (and also other) consumption is generally expressed in gallons per capita of population per day, averaging the days for an entire year—obtained by dividing the average total domestic consumption for one day by the total population. A more useful plan is to divide such average total domestic consumption by the number of consumers (i.e., those living in residences in which the public supply is used); but it is difficult to ascertain the number of such users, and this practice is by no means common, but is increasing.

TABLE NO. 1

CONSUMPTION OF WATER IN IOWA CITIES IN 1914

Name of City.	Estimated Population in 1914.	Estimated Per Cent of Population Reached by Mains.	Estimated Per Cent of Population Using Water.	TOTAL GALS. PER DAY.		Per Capita of Water Users Less Large Consumers †
				Per Capita of Total Population.	Per Capita of Water Users.	
Sioux City.. ..	50,000	80	66	56	84	84
Davenport. .	45,500	100	85	88	103	103
Waterloo ..	32,000	82	75	42	55	55
Burlington. ..	24,000	95	90	95	105	97
Fort Dodge.	20,000	90	90	55	61	
Muscatine..	18,000	69	50	71	141	134
Keokuk	16,000	90	75	83	111	107
Iowa City.	14,500	80	75	120	160	154
Mason City.	16,000	81	56	55	98	66
Marshalltown	14,000	100	99	114	115	60
Boone ...	12,000	40	40	94	235	138
Creston .	8,000	75	45	75	167	154
Charles City	7,000	70	64	47	73	49
Cedar Falls.	7,000	95	78	67	88	88
Cherokee.	5,000	90	67	36*	43*	51*
Red Oak.	5,000	100	100	60	60	
Le Mars. .	5,000	95	90	150*	167*	160*
Clarinda.	4,500	75	67	30*	44*	14*
Waverly.	4,000	75	50	41*	82*	62*
Spencer . .	4,000	67	50	38*	75*	75*
Sheldon	3,200	78	62			
Algona. .	3,200	78	62	28*	45*	45*
Osceola .	2,800	43	32	14*	44*	
Sac City	2,700	74	74	28	38	38
Storm Lake.	2,500	95	75	80*	106*	77*
Villisca.	2,300	35	35	12*	34*	30*
Rockwell City.	1,800	49	49	47*	93*	61*
Sibley. .	1,700	100	35	34*	83*	83*
Spirit Lake	1,700	59	41	41*	100*	80*

* Consumption estimated only.
† In calculating this column the total consumption was diminished by subtracting from it the amounts used by railroads and large manufacturing consumers.

The consumption per user or per capita of population may generally be greatly diminished by restricting waste by the use of meters or other means. Where these are introduced there is generally a drop in total consumption during the time of introduction, but the increase due to the growth of the city soon begins at about the same rate as before, although the quantities continue less than would have been the case had meters not been used.

The per capita consumption in different cities varies widely. Very few reliable figures for domestic consumption are available, most cities knowing only the total consumption for all

uses, and many having only a very indefinite idea of this. In Table No. 2 are given the domestic rates for 14 cities which metered practically all their consumption, and for one, for

Table No. 2

ANALYSIS OF WATER CONSUMPTION IN 1914 IN CITIES ALMOST WHOLLY METERED

From report of Committee on Water Consumption of American Water Works Association

City.	Population.	Per Cent of Services Metered.	Per Cent Minimum Night Rate is of Day Rate.	Consumption; Gallons per Capita per Day.					Total Unaccounted for	
				Total.	Metered Industrial.	Metered Commercial.	Metered Public.	Metered Domestic.	Per Cent.	Gals. per Capita.
Milwaukee, Wis	430,000	99.4	62.5	111.4	41.4	32.0	5.6			
New Orleans, La.	360,000	99.7	.	57.2	*	13.0	3.2	16.8	37	24.2
Rochester, N. Y.	250,000	99.5	42.0	95.0	18.3	12.2	4.8	31.2	30	29.5
Utica, N. Y.....	95,000	99.3	....	78.7	41.9		2.2	20.9	18	13.7
San Diego, Cal.	85,000	100.0	...	80.6	7.2	12.1	9.6	36.1	19	15.6
Wilkinsburg, Pa	80,000	99.0	....	107.0	59.6	.	2.4	25.5	18	19.5
Buffalo suburbs.	50,000	100.0	...	131.5	98.5	.	..	8.8	18	24.2
Lexington, Ky	40,000	100.0	42.5	62.1	18.0	16.0	8.4	16.8	5	3.0
Saginaw, Mich†.	39,600	4.1	. .	333.7	22.3	7.4	..	45.0		
Madison, Wis..	27,000	99.2	14.1	79.0	6.1	6.5	3.4	35.3	36	27.7
Oak Park, Ill.	26,000	100.0	44.5	69.0	8.6	1.7	2.5	52.4	6	3.8
Pine Bluff, Ark..	16,000	100.0	83.8	71.6	*	7.5	..	12.3	72	51.8
Elyria, O.....	16,000	100.0	38.3	121.1	47.0	7.2	5.6	38.8	18	22.5
Corning, N. Y.	14,900	99.0	16.0	83.3	16.7	7.2	1.4	58.0		
Monroe, Wis...	3,000	100.0	56.0	79.0	38.5	..	..	33.3	9	7.2
Mean.........			44.4	87.6	29.4	10.5	4.5	29.7	24	

* Included in commercial.
† Introduced as an extreme case of comparatively unmetered service with high consumption.

comparison, that metered "very little." These are seen to vary from 8.8 gallons to 58.0 per capita per day. The figures in this table are probably more reliable than those most commonly found in consumption tables. Taking the 14 cities in

which 99 per cent or more of the services are metered, and which are therefore most reliable, we find the domestic consumption to average 29.8 gallons per capita per day. In general, it is probable that the domestic consumption of most cities lies between 20 and 50 gallons per capita per day, with 15 and 100 the extremes, except for a few cases of excessive waste. Where domestic consumption exceeds 30 or 40 gallons, the surplus is generally waste (see Art. 6).

Industrial consumption has little relation to population, except that, while the nature of the industries in a given community largely occupied in manufacturing remain the same, total amount of manufacturing and consequent use of water therein increase at about the same rate as population. But no comparison in this respect can be made between communities in which manufacturing conditions differ.

Commercial consumption bears a more uniform relation to population. Comparatively few figures are available, but apparently between 5 and 15 gallons per capita of total population is common.

Public use varies with the length of sewers flushed, area of streets sprinkled and flushed, number of fountains and drinking troughs, and especially with the amount used in schools and other public buildings, in which millions of gallons are often wasted daily. The amount used in extinguishing fires in a year is generally less than 1 per cent of the supply and may be disregarded. Many cities that meter all other supplies do not meter that furnished free for city uses, and this amount is thus often included in the "unaccounted for" water. Public use probably lies ordinarily between 3 and 15 gallons per capita per day, increasing as the city grows, because sewerage and other public services are more general in a large than in a small city.

Unaccounted for water may include and total up to almost anything. As used here, it means the water which is believed to be delivered to the distribution system, less that which, with all services metered, is recorded by such meters. That is, it is the water that escapes underground from leaks, or goes

through the meters without being recorded, and possibly may include an error in the amount believed to be delivered to the mains. The last may be inaccuracy in the meter through which the entire supply passes at distributing reservoir or pumping station (where there is such a meter, as there always should be), or erroneous estimate of slip of pump. Most pumps contain counters that record the number of strokes. Theoretically a given pump delivers a certain amount of water at each stroke; practically the amount is less than this by at least 1 or 2 per cent in a new pump, by 5 to 10 per cent in most pumps after a year or two of service, and has been found as high as 50 to 75 per cent in badly worn pumps. This difference between theoretical and actual pumpage is called "slip." The service meters (those on the lines delivering water to individual buildings) are generally correct within 1 per cent when new, but they may under-register when worn, clogged with sediment, or broken, and thus fail to record all the water passing through them.

The escape of water underground is a more serious matter, because it benefits no one—in fact often does damage, and is very difficult to remedy. It is generally the leakage from joints, or from cracked pipe or other defects in the distribution system. (See Art. 6, "Waste of Water.") It is approximately proportional to the length and size of the pipe line, and is frequently expressed in terms of "inch-mile" per day, that is, the number of gallons of leakage per day divided by the product of the number of miles of pipe and its diameter in inches. Underground leakage is seldom less than 100 gallons per inch-mile per day, and may amount to 10,000 or more. As the mileage of line varies in a general way with the population served (see Table No. 44), leakage also may be expressed in terms of gallons per capita per day. It is seldom less than 2 gallons per capita per day, and may run as high as 50.

Total Consumption. Combining these several classes of consumption, and using the rates more commonly found, we have the following:

Domestic	20 to 50 gallons
Commercial	5 to 15 gallons
Public	3 to 15 gallons
Unaccounted for	2 to 20 gallons
Total	30 to 100 gallons

In addition there may be considerable amounts used for industrial purposes; although where this is large, the industries may obtain their own supplies rather than purchase from the public supply. Some engineers, planning for thirty to fifty years ahead for large cities, have considered 150 gallons as a safely high estimate, including industrial with all others.

Having made as intelligent estimates as possible of each of the above and the total daily consumption per capita, the total consumption for the year will be 365 times this, times the estimated population. If it is a matter of extending an existing system, careful measurements of actual consumption should be made over as long a period as possible, classifying uses as above, finding night consumption (which is largely leakage), and determining as far as possible all local conditions that affect consumption; and on the basis of this study and the principles above outlined, estimate the future consumption.

Art. 5. Consumption for Short Periods

The consumption referred to in the previous articles is the *average* daily consumption. The actual rate of consumption will vary from month to month, day to day, and hour to hour. The maximum consumption will generally be in the dryest summer weather in suburban sections, where it is used in lawn and street sprinkling; but frequently occurs in very cold weather in urban districts, due to waste through faucets at night to prevent freezing. The monthly rate will generally exceed the annual by about 15 to 30 per cent. No figures are available for maximum rates for the several uses. The industrial rates will probably be quite constant, and the commercial rates

fairly so, except for the cold-weather maximum. Public use will increase in summer because of use for sprinkling streets and parks, for fountains, etc. "Unaccounted for" will be quite uniform throughout the year. Most of the variation will be in the domestic, but less in the monthly than in the daily and hourly rate.

The day of the week makes little difference in the rates of commercial, industrial, and public use, except that the two latter drop to almost nothing on Sunday, and commercial also except for that used in hotels, drug stores, stables, and a few other establishments. School use is low on Saturday also. Domestic use is generally greatest on Monday (wash day). As an approximate average it may be said that the maximum daily rate in any given city may be 50 to 100 per cent greater than the average daily for the year.

The maximum hourly rate will probably be that when a large fire occurs coincident with a general maximum. All services except "unaccounted for" are much higher between 5 A.M. and 7 P.M. than at night. In most cities the rate begins to rise about 4 A.M., reaching a maximum about 8 or 9; dropping considerably between 12 and 1, and remaining high between 1 and 6; from which hour it continues falling until sometime between 10 and 2, depending upon the local habits as to bedtime. From this last hour until about 4 or 5 the consumption is a minimum; in some cities it is almost exclusively leakage in distribution system and leakage and waste in house plumbing.

The rate in a large city is not greatly increased by a small fire, and even the great Baltimore fire did not double the rate of consumption. In a small city or town, however, the amount used at a fire may be several times the total consumption for a day, and the maximum water consumption per minute during the fire may be many times the rate per minute of ordinary consumption. Each fire stream generally throws between 175 and 250 gallons per minute, depending upon pressure and size of nozzle. The number of fire-streams of such size that should be provided for in American cities of various magnitudes is considered by different authorities to be as follows:

Table No. 3

NUMBER OF FIRE-STREAMS REQUIRED SIMULTANEOUSLY IN CITIES OF VARIOUS MAGNITUDES

Population.	Freeman.	Shedd.	Fanning.	Kuichling.	Gallons per Minute by Freeman.	Gallons per Minute per 1000 Pop.
1,000	2–3	......	......	3	350–750	350–750
4,000	(4–6)	......	7	6	700–1500	175–375
5,000	4–8	5	.....	6	700–2000	140–400
10,000	6–12	7	10	9	1050–3000	105–300
20,000	8–15	10	......	12	1400–3750	70–190
40,000	12–18	14	......	18	2100–4500	52–110
50,000	(14–20)	......	14	20	2450–5000	49–100
60,000	15–22	17	......	22	2625–5500	44– 90
100,000	20–30	22	18	23	3500–7500	35– 75
150,000	(24–36)	......	25	34	4200–9000	28– 60
180,000	(27–40)	30	......	38	4725–10000	26– 55
200,000	30–50	... ..	.....	40	5250–12500	26– 62

The National Board of Fire Underwriters, in December, 1916, adopted the following as the standard requirements on which they base fire insurance:

Table No. 4

REQUIRED FIRE FLOW, NATIONAL BOARD OF FIRE UNDERWRITERS

Population.	Required Fire Flow, Gallons per Minute for Average City.	Population.	Required Fire Flow, Gallons per Minute for Average City.
1,000	1000	28,000	5,000
2,000	1500	40,000	6,000
4,000	2000	60,000	7,000
6,000	2500	80,000	8,000
10,000	3000	100,000	9,000
13,000	3500	125,000	10,000
17,000	4000	150,000	11,000
22,000	4500	200,000	12,000

Over 200,000 population, 12,000 gallons a minute, with 2000 to 8000 gallons additional for a second fire.

In residential districts the required fire flow depends upon the character and congestion of the buildings. Sections where buildings are small and of low height, and with about one-third the lots in a block built upon, require not less than 500 gallons a minute; with larger or higher buildings up to 1000 gallons is required, and where the district is closely built, or buildings approach the dimension of hotels or high value residences, 1500 to 3000 gallons is required, with up to 6000 gallons in densely built sections of 3-story buildings.

TABLE NO. 5

MONTHLY RATES OF WATER CONSUMPTION IN SEVERAL CITIES

Average Percentage of Annual Rate

City.	Population.	Duration of Record, Years	January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Average Daily Consumption.
Rock Island, Ill	13,595 to 15,020	6	98 2	98 5	91 7	89 0	90 7	105 2	112 3	114 3	114 0	105 2	99 6	94 8	143
Average of 13 New England cities and towns	....	...	87.2	89.0	88 6	89 7	99 8	114 0	123 0	113 5	109 4	103.0	92.1	88 7	
Brooklyn, N. Y.	690,000 to 940,000	10	97.1	100 1	98 4	95 9	97 5	102 4	104 3	103 3	104 4	100 2	96 8	99 4	70
Rockford, Ill..	24,000	2	87.9	98 3	88 3	97 7	101 0	105 6	106 3	109 7	109 9	99 2	98.8	94 3	94
Taunton, Mass..	27,000	4	87 5	101 9	91 7	94 4	103 2	114 0	116 4	112 4	101 0	98.9	91.9	86 5	42
Newton, Mass.	27,590	18	85.0	86 0	84 0	89 0	98 0	120 0	125 0	116 0	112 0	99 0	94 0	92 0	63
Philadelphia, Pa.	1,175,000	1	89 9	85 0	82 4	89 6	99 9	110 4	111 7	110 8	106 4	109.8	98 3	106 7	152
A fair average will be about.....			87	90	87	90	98	115	121	115	110	101	94	92	

TABLE No. 6

MAXIMUM RATES OF CONSUMPTION

City.	Duration of Record.	Population.	Daily Consumption, Year in Question.	Monthly Maximum.		Weekly Maximum.		Daily Maximum.		Hourly Maximum.	
				Month.	Rate Per Cent of Average.	Month.	Rate Per Cent of Average.	Date.	Rate Per Cent of Average.	Hour.	Rate Per Cent of Average.
Taunton, Mass.	3 years	27,000	42	July	128	......	......	. .	......	......	......
Rockford, Ill.	2 years	25,400	96 5	August	117	......	......	.. ...	......	......	......
Philadelphia, Pa.	2 years	1,175,000	155	July	112	......	......	Sept.	124	......	......
Detroit, Mich.	3 years	260,000	150	. ..	125	......	140	. ...	150	......	178
Attleboro, Mass. and Woonsocket, R. I.	1 year	7,577 20,830	40 5 26 1	. . .	122	......	134	......	155	8 A.M.	333
Brooklyn, N. Y.	10 years	700,000	65	July	108	......	......	Feb. 8	127	......	......
Reading, Pa.	2 years	74,400	87 1	......	......	......	......	July 9	131	......	......
Rock Island, Ill.	......	.. ...	121	August	128	......	......	......	......	......	......
Boston (Mystic supply)	1 week	117,000	73 6	......	......	......	......	. ..	......	8 A.M.	141
Newton, Mass.	18 years	10,000 to 30,000	60.1	August	167	July	226	July	296	......	......

The amounts given above probably would be demanded only at intervals of many years; but when needed, they may be necessary to prevent the entire destruction of the city.

Districts. The figures for fire streams apply to districts as well as to cities. That is, the high rate recommended for a town of 1000 population should be provided for a district containing that population in a large city where the average rate for the city is only one-tenth as great. This is an important consideration in determining the sizes of the pipes of the distribution system.

It is also important to consider in this connection the districts where large industrial consumption will occur, such as railroad yards, where the public supply will be used for filling locomotive boilers. As to domestic consumption, Dexter Bracket found in Boston a rate of 59 gallons per capita in the highest-cost apartment houses, 46 in first-class apartments, and 16.6 in the poorest class.

Art. 6. Waste of Water

There are very few cities in which a yearly average of 75 gallons per capita per day is not sufficient for all uses, and in most cities where the consumption exceeds this it might be brought within this limit by proper treatment. This being the case, it is evident that there must be a great amount of water wasted in many cities. In New York City this appears to be about 40 gallons, in Detroit 50, and in Philadelphia more than 140 gallons per capita daily. If the supply is pumped, this means increased expense for coal consumption and enlargement of pumping-plant; if the supply is from wells or reservoirs, it hastens the day when new wells must be sunk or new drainage areas sought; and in any case it means that either the mains must be unnecessarily large or the pressure will be decreased below that desired. Millions of dollars are being spent by many of our larger cities to increase their supply in order that two-thirds of it may be wasted.

This waste is either intentional, careless, or through ignor-

ance. Under intentional waste may be classed the opening of faucets on winter nights to prevent freezing of poorly located plumbing; the lavish sprinkling of lawns and streets in summer; and unnecessary amounts used in automatic sewer flush-tanks, as well as in automatic water-closet flushes in hotels and office buildings. Carelessness usually takes the form of non-attention to leaky house-fixtures, thousands of which can be found in almost any city. More or less leakage in every system defies detection, being probably the result of very slight leaks in thousands of joints, in fire hydrants, stop-valves, corporation cocks, and other portions of the distributing system. This has been found to be 5 to 10 per cent of the consumption in some cities.

The winter waste due to the first-mentioned cause is shown in Table No. 5. In Rock Island this was greatest in January and February; in New England, Brooklyn, and Rockford, Ill., in February. In Rockford the February consumption is seen to be about 8 per cent greater than that of the other two winter months. Daily records would show this waste much more prominently. At Newton, Mass., during a twenty-year period the winter weekly maximum has fallen in December four times, in January four times, and in February nine times; the greatest excess being in the second week of December, when the average consumption for the week was 122 per cent of the annual.

The summer waste is more prominent, the average consumption during June, July, and August running from 4 to 15 per cent above the yearly average.

The waste of water by the city in flushing sewers by automatic tanks is in some cases most excessive. From 500 to 800 gallons per day per tank should be sufficient for the proper maintenance of a lateral sewer; but carelessness or ignorance has been known to cause ten to thirty times this amount to be used. The water company of Racine, Wis., found the 111 flush tanks of that city using 1,436,429 gallons per day, an average of about 13,000 gallons each—an actual waste of certainly more than 1,000,000 gallons per day. Street-

sprinkling is also a common cause of much waste, one Massachusetts city using 7 per cent of its entire supply for this purpose.

The loss through leaky house-fixtures is enormous. In Philadelphia, of 782 appliances in a certain section containing 539 population, 22 were leaking slightly and 32 were running continuously; and it was found in one district that 63 per cent of the water furnished was wasted by 17 per cent of the consumers, and in another 86 per cent was wasted by 7 per cent of the consumers, this being not lavish use, but absolute waste.

A leaky faucet may waste 75 to 300 gallons per day, and a leaky ball-cock may easily permit 1000 gallons per day to pass it. Unless this leakage inconveniences the householder he will seldom repair the defective fixtures if not compelled to. The greatest amount of leakage of this class is generally found in rented houses, particularly of the poorer class, where the plumbing is cheap, and money for repairing a leak is spent grudgingly.

Leakage from mains and house connections has been determined by leakage surveys (generally by use of the "pitometer") in a number of cities, Washington, D. C., having perhaps gone into the matter most thoroughly, and maintaining for several years a corps of men for this purpose alone. During one year this squad found leaks from which there was escaping a total of 9,560,635 gallons a day, about one-half of which was from leaking service pipes, about 15 per cent from joints in the mains, and the remainder from broken mains, cracked valve-casings, leaking fire hydrants, etc. It seems to be impracticable to entirely eliminate leakage from joints. Leakage from cast-iron mains has been found varying in different cities from 150 to 10,000 gallons per day per inch of diameter per mile. In a new, well laid pipe line it should not exceed 150 to 500 gallons per inch-mile.

Waste of water should be prevented or paid for, for it is furnished at a considerable cost and it is not just that a few should carelessly waste what others have to pay for. Each consumer should be required to pay for the amount of water that enters his premises, which should be measured. There is no more reason why water should be sold at so much a fixture

than why gas or electric light should, or than why food should be sold at so much per individual per year, instead of by the pound, bushel, or other measure.

Art. 7. Forecasting Future Consumption

As stated previously, each structure or feature of a water-works system should be designed with a view to being of ample capacity for some years to come. Water mains cannot be too large for effectiveness, nor can the catchment area. But an intercepting reservoir should not be larger than is necessary to equalize over a number of years the variable run-off from the catchment area; no one pump should be larger than sufficient to provide the present average consumption of a day with about five or six hours' operation; and purification plants should be built with only 10 to 20 per cent surplus capacity (beyond that needed for cleaning the beds, see Art. 18), since new units can be added easily. The land on which the filters and other structures rest should, however, be sufficient to provide for expansion for many years to come.

The penalty of estimating too low on the pipe lines is excessive loss of pressure due to pipe friction, or else the necessity of building a duplicate line sooner than is economical. In the case of pumps, the penalty is the same, or possibly a breakdown of the plant caused by working it too hard. If the catchment area or reservoir is too small, a dry spell will bring a shortage of water, requiring another supply to be obtained—possibly a less desirable emergency supply.

The general basis for calculating sizes of pipes is an estimate of the future growth of the city or section to be supplied through the pipe in question; then by calculation compare the cost of two plans, one providing for the consumption x years ahead and then duplicating the line, the other providing at once a line whose capacity is that of the other two combined; the cost of each plan being taken as the principal and compound interest computed to that future date at which it is computed that the capacity of either plan will be reached. (This method

of calculation is not strictly correct, but the error involved is less than the probable error in estimating the future growth of city, cost of laying pipe, etc.). Let C = cost of laying one large pipe (second plan) and c that of a pipe half the capacity (first plan), and r the value at compound interest of $1 computed for the time between the laying of the first and second pipes by the first plan. Then $r=\frac{c}{C-c}$. Assume the rate at which money is borrowed for building the pipe system (say 4 per cent) and look in compound interest tables for the value of r at that rate. The year corresponding will be the time for which the first pipe line may be calculated and still leave the cost of the two lines (first plan) not greater than that of one line. If the second line would be required sooner than this, the two lines will prove more expensive than one having their combined capacity. If it would suffice for a longer time, the two lines will be the cheaper. For example, if we compare the present construction of an 18-inch line, to be duplicated later with another of the same size, with a 24-inch (which has double the capacity of an 18-inch) built at once, and assume that the first costs $2.87 a foot and the latter $4.61; then $r=\frac{2.87}{1.74}=1.65$, which, at 4 per cent interest, corresponds to a little less than thirteen years. Then it is cheaper to build an 18-inch line if that will suffice for thirteen years or more, and duplicate it when the capacity of the first is reached, than to build a 24-inch at once. If two 6-inch lines be compared with one 8-inch on the same basis, it is found that the first 6-inch must have ample capacity for at least twenty-four years to be more economical than an 8-inch. Or, in other words, if a 6-inch will not give capacity for twenty-four years to come, then an 8-inch (or larger) should be put in at once. There are other things to be considered, such as the better pressure resulting from the large pipe, the deferring of the tearing up of the street for laying another line and the less total length of joints to leak in one line than in two, and the smaller number of repairs probably required. On the other hand, it is possible

that the development of the city or district served by the pipe may not be so rapid as calculated, or may take another direction from that anticipated, when construction of the second pipe line may be deferred or the second condition can be met by taking another route. The possibility of pipe costing more or less ten or twenty years from now should be considered also.

The consumption for which pumps, reservoirs, purification plants, etc., should be designed is determined in something the same way, but will be considered each under its appropriate head.

It is seen that, to forecast consumption of all kinds (except perhaps industrial), we must first forecast population. This may be done by one or more of the several methods in common use. (See Chapter I of the author's work on "Municipal Engineering Practice.") If possible, the present consumption for each of the several uses by the city in question should be determined, the conditions of each of such uses determined, and the probable tendency toward an increase or decrease in per capita rate for each use. Using with judgment all the facts available, a rate per capita for the city as a whole and for each district is estimated for each of the next five or more decades; and the city and district populations as forecasted multiplied by such consumption rates will give the total consumptions. These may then be plotted or tabulated for each of the future fifty years.

CHAPTER III

REQUISITES OF A SUPPLY. QUALITY

ART. 8. WATER AND ITS IMPURITIES

CHEMICALLY pure water is a mixture of hydrogen and oxygen and contains no other matters in either solution or suspension. Such water is found only in a chemical laboratory; for most solids and gases are more or less soluble in water, and it has great avidity for many of them; and it mixes readily with most other liquids. Water is always necessarily in contact with some solid substance, and generally with air also, and may absorb some chemical elements from each of them. The nearest approach to chemically pure water practicable for laboratory work is distilled water kept enclosed in porcelain or glass.

The term "pure" is popularly used in connection with water to indicate freedom from matters that render it objectionable from the point of view of the speaker. It is very desirable for purposes of accuracy and definiteness of meaning that more precise terms be used. The word *safe* has been proposed for describing potable water free from pathogenic or toxic matters. Other terms of established usage are *clear*—free from suspended matter; *colorless*—free from coloring matter in solution or semi-solution; *acid*; *alkaline*; *hard*—containing salts that precipitate soap; *soft*—free from such matters.

Polluted is used to indicate water that has received impurities from other than natural sources, especially waste waters from communities of human beings; *normal* describes waters which have not been polluted.

The various matters found in water are classified by M. F. Stein as follows:

SUBSTANCES OF MINERAL ORIGIN	SUBSTANCES OF ORGANIC ORIGIN
In suspension	
Clay and inorganic soil wash.	Organic soil wash. Decomposing organic matter.

SUBSTANCES OF MINERAL ORIGIN	SUBSTANCES OF ORGANIC ORIGIN
In pseudo-solution (extremely fine particles in suspension)	
Silica Alumina Iron oxide	Colloidal organic wastes Vegetable color Organic acids
In solution	
Bicarbonates, Carbonates, Sulphates, Chlorides, Nitrates } of { Calcium, Magnesium, Sodium, Potassium	Vegetable color Organic acids Soluble organic wastes Ammonia Chlorides Nitrites Nitrates
Bicarbonates, Sulphates, Hydroxide } of Iron	Carbon dioxide, Hydrogen, Hydrogen sulphide, Methane } gas
Mineral acids	
Carbon dioxide, Oxygen, Nitrogen } gas	

There are also living organisms, both animal and vegetable, of all sizes down to the sub-microscopic.

Sources of Impurities. A drop of rain falling through the air absorbs from it carbonic acid, oxygen, nitrogen, and other gases, and dust, bacteria and other minute matters attach themselves to it. Rain which, after falling, flows over the ground, generally softens this and removes particles of it carried in suspension, and may take into solution many kinds of mineral and organic matters.

The water which percolates into the soil and issues therefrom elsewhere in the form of springs or of a general seepage into water courses absorbs carbonic and other acids from the decaying vegetable matter therein (roots, etc.). These acids dissolve some minerals not soluble in pure water, while other minerals and salts are dissolved by the water as such.

In passing over or through the primary rocks—granite, basalt, gneiss, etc.—the slightly acid water leaches from them minute quantities of alkalies and alkaline earths, taking them into solution as bicarbonates. Silica, alumina, and iron oxide are removed in a colloidal form. These waters, called primary waters, are characterized by proportionately large amounts of

salts of sodium and potassium and small amounts of salts of calcium and magnesium, and by silica, alumina and sometimes iron in colloidal state.

Water passing over and through secondary formations—limestone, dolomite, sandstone, shale; sodium, calcium, and magnesium chlorides, gypsum, etc.—dissolves out calcium and magnesium, taking them into solution as bicarbonates (these give the water the property of temporary hardness). Sodium, calcium, and magnesium are removed as chlorides and sulphates (these make the water permanently hard). Iron is removed as a soluble ferrous bicarbonate (when the water is low in oxygen). These are called secondary waters.

Water also receives salt from the air, carried inland from the sea as spray or mist, which falls directly upon the water, or upon the soil, from which it is removed by rain. This is so general that the amount of salt in a water that receives no salt or very little from other sources varies with its distance from the ocean, and the amount in normal water originating in a given district is practically constant. For reasons to be described later, the salt content of water is an important consideration, and the normal salt content of the waters of each district has been determined for a number of coast States. A map showing isochlors (lines passing through districts in which the normal salt content of the waters is the same) is shown in Fig. 1.

Water generally carries in solution carbonic acid gas, oxygen (uncombined with hydrogen), and nitrogen. Each is derived from the air, and the first chiefly from decaying organic matter in and on the soil. Plant life growing in water in the presence of light absorbs carbonic acid and gives out oxygen; and also, in another phase, takes in oxygen and gives off carbonic acid, this taking place more freely in darkness than in light. Hydrogen sulphide is received from decaying organic matter.

Water also may receive impurities in the form of drainage of cities, coal mines, oil and salt wells, and other industrial waste matters. From coal mines come sulphuric and sul-

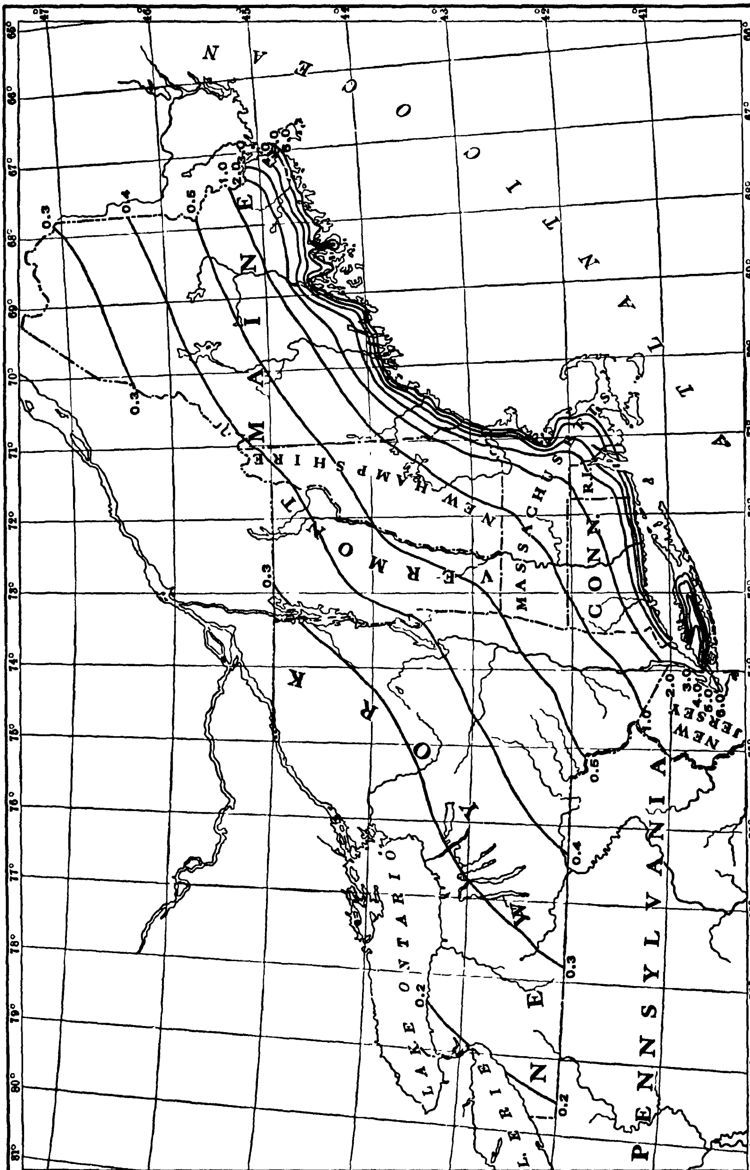


FIG. 1.—Normal Chlorine Curves (Isochlors) of New England and New York. From map prepared by U. S. Geological Survey.

phurous acids. Acids are contributed by steel mills, tanneries, and other industries; organic matters by paper mills, slaughter houses, and breweries. From sewage are derived large quantities of nitrogenous matter (which absorbs oxygen from the water to form nitrites and nitrates), hydrogen, methane, ammonia, and carbonic acid; also chlorine, which is found in all urine, about .04 of a pound being excreted daily by the average person.

Suspended matters are seldom present in more than the most minute quantities in water issuing from the ground, any which it may have once contained being filtered out by passage through the fine pores of the soil, or deposited by gravity or surface adhesion to the soil particles during its slow progress underground. But on emerging, some of the soluble matters may be rendered insoluble and go into the suspended form. Thus ferrous carbonate ($FeCO_3$), when oxygen is supplied by the air, changes to the insoluble ferric form (Fe_2O_3). Suspended matters are also supplied abundantly in the form of clay, comminuted leaves and other organic matters washed from the ground surface by rain. Dust, both mineral and organic, falls into water from the air, the latter including pollen from vegetation, dried bodies of insects, etc.

An important class of suspended matters is that of the microscopic living organic matter. Even rain water is found to contain algæ, usually some protococcus form. Water directly from the ground contains none, with rare exceptions; and water from deep wells seldom does, although crenothrix (an iron bacterium) sometimes grows abundantly on steel well casings. It is in quiet waters that plankton * develop in abundance, and they are carried in streams fed by such waters or communicating with them. Most of the forms are vegetable, but animal forms are found in rivers polluted with organic matter. The organisms other than bacteria (which will be discussed farther on) that are of most interest in connection with public water supplies are the algæ.

Algæ are microscopic plants whose cells contain chlorophyll.

* Plankton are the minute animals and plants that float free in water.

They can develop only in water in the presence of light, and therefore within a few inches of the surface in turbid water, but to a depth of 25 feet in very clear water. They live upon organic matter. They require oxygen, and if they cannot obtain it from the water, they take it from the organic matter, producing anaerobic decomposition, or putrefaction. Algæ in themselves are not injurious to health, although if taken in large quantities they may temporarily cause intestinal troubles. But they produce odors and tastes that are decidedly unpleasant. (See also arts. 9, 13 and 38.)

Bacteria are microscopic, one-celled fungi, generally having power of motion. Probably no surface water is free from them, and there may be millions in a cubic centimeter. Most of them are harmless, and many assist in the decomposition of organic matter and are essential to the self-purification of water. A few species, however, are capable of producing disease if taken into the stomach. The natural habitat of the latter is the intestines of animals; but if carried into water with the excreta, they may survive for days and even weeks. The most common diseases so caused are typhoid fever, cholera, dysentery, diarrhœa, and other intestinal disturbances, each caused by a special bacterium. There are, however, some species abounding in the intestines which do not cause disease. Among these are the Coli bacilli (called B. coli for short), which seldom originate outside of the intestines of man and the higher animals, are much more abundant in excreta-carrying water (sewage) than are B. typhosus (the cause of typhoid) or the other pathogenic bacteria, and therefore, since they are readily identified, are used as the test for sewage pollution. The pathogenic bacteria are the only matters contained in ordinary sewage which are harmful in the quantities in which they can exist without making the water objectionable to the senses.

Sewage contains considerable amounts of organic matter in various stages of decomposition: the soluble products of such matter already referred to; and particles of grease, hair, fibers of wood, cloth, and paper, and other miscellaneous suspended matters.

Art. 9. Qualities Caused by Impurities

Impurities in water in most cases impart to it certain characteristics or qualities, physical or chemical. The most important of these are turbidity, color, taste, odor, alkalinity, acidity, and hardness.

Turbidity is the presence in water of visible suspended matter. It is caused chiefly by clay, fine sand, or other earthy materials, that are carried in suspension by flowing water, since it is seldom that other suspended matters are present in sufficient quantities to produce appreciable turbidity. A turbidity of 5 parts per million (see Art. 12) is barely discernible; 100 parts gives a very cloudy appearance; 1000 parts makes water practically opaque; 10,000 is practically the maximum for most waters and kinds of suspended matters. Five parts may be taken as the limit for acceptable drinking water.

Color. The visible suspended matters that cause turbidity have color, and therefore the water appears to be colored. Actual color of the water itself, however, is caused by matters in solution or colloidal solution, the exact chemical nature of which is not known. Coloring matter is most commonly in the form of tannates, gallates, or organic acids derived from decaying vegetable matter. Swamps are the most important sources of color in water. Water from a clean watershed seldom has a color higher than 30, platinum scale (see Art. 12), but that of stagnant water in swamps often runs as high as 300, and sometimes 500 or 700. Many matters causing turbidity have the power of drawing to themselves by adsorption any color that may be in the water; therefore turbid waters seldom have appreciable color.

Water in most ponds or lakes has a brownish color, which may be as dark as that of weak tea. Freshly fallen leaves and vegetable matter give a greenish tinge to the brown, and the hue changes periodically. The amount in a given stream varies with the proportionate amounts of swamp or stagnant water and of clear water reaching it, which vary from time to time.

Taste and Odor. These are closely related in the sensations produced, one sometimes being mistaken for the other, and many so-called tastes are really odors. Almost every pollution gives a taste, and many give an odor to water, but in the minute quantities in which most impurities are present neither can be detected. Heating water usually intensifies the odor. Most of the bad odors in water are due to organic substances in solution or suspension or to microscopic organisms. "Brown-colored waters invariably have a sweetish vegetable odor, and the intensity of the odor varies almost directly with the depth of the color. Both color and odor are due to the presence of certain glucosides, of which tannin is an example, extracted from leaves, grasses, mosses, etc. In addition to the odor, these substances have a slight astringent taste. Colorless waters containing organic matter of other origin may have vegetable odors, but they are usually less sweetish and more straw-like or peaty. Akin to the vegetable odors are the earthy odors caused by finely divided particles of organic matter and clay." (George C. Whipple, "The Microscopy of Drinking Water.")

Decomposing vegetable matters may produce decidedly unpleasant odors. Disagreeable odors usually are caused by decaying animal matter. Offensive odors are usually due to sewage or other gross pollution.

The odors most commonly found in drinking water are those due to living organisms in the water, produced in most (possibly in all) cases by compounds analogous to the essential oils which are secreted by vegetable growths. They are most noticeable when the breaking up of the organism causes the oil globules to be scattered through the water. These growths (mostly algæ) develop most luxuriantly in still water, and reservoirs are the part of a public water supply where such odors generally originate. It is believed that only the floating organisms yield such odors. Some of the littoral plants possess a vegetable or fishy odor, which, however, is imparted to the water only when the plants are crushed.

Hardness. Carbonates of calcium and magnesium give water

a quality known as temporary hardness. Sulphates of magnesium and calcium and chlorides of sodium and calcium make water permanently hard. Boiling expels carbonic acid, and the carbonates are thrown out of solution—hence the term "temporary." Permanent hardness is not removed by boiling. The same water may be both temporarily and permanently hard.

Art. 10. Effects of Impurities

The effects of impurities in water may be divided into two classes, according as they are or are not morbific.

Hard water forms scale in boilers and necessitates the use of more soap in washing. Sulphate scale (deposited by permanently hard water) is worse than the carbonate, because it is denser and harder. The cost to the U. P. R. R. in one year for cleaning and repairing locomotive boilers because of scale was 8 cents per locomotive-mile. Scale is said to be the most frequent cause of steam-boiler explosions. It is a poor conductor of heat, and when it forms on a tube or plate it interferes with the transference of heat to the water and may cause the metal to be heated to a plastic temperature, when it is ruptured by the steam pressure. Even when this does not occur, the steam-creating power of the fuel is lessened, and the cost for fuel is consequently increased. Experiments indicate that scale $\frac{1}{32}$ of an inch thick causes a waste of 3 or 4 per cent of the fuel, and $\frac{1}{4}$-inch scale on all the tubes may increase the coal bill 25 per cent or more. Scale may also clog feed pipes, water-gage and steam-gage connections, and other boiler-room piping. The same troubles, although less in intensity, may be caused by scale in the boilers of domestic steam-heating plants.

Soap is, of course, used with but a small percentage of the water consumed by a community (estimated at 2 gallons per capita per day in Sacramento), but Whipple ("Value of Pure Water") estimates that 1 pound of soap is used up in softening from 100 to 225 gallons of water having a hardness of 20 parts per million, and in softening from 15 to 34 gallons when the hardness is 200 parts, the amount depending upon the kind

of soap used. From this can be computed the actual cost to a community, in soap consumed, of hardness in water. Hardness may be an advantage in some cases; for instance, calcium sulphate in the Trent river is largely responsible for the quality that makes Bass ale popular.

Carbonic acid in water dissolves many metals more or less freely, lead, zinc, brass, and iron being those commonly attacked by public water supplies. These metals are seldom present in water in sufficient amount to have any except a morbific effect, however. The qualities of water which cause it to dissolve metals are not known exactly; but it has been learned in a general way that these are softness, acidity, dissolved gases, and the presence of much chloride or nitrate. The only sure decision must be based on an actual test of the metal solvency of the water in question. Corrosion is destructive to steel boiler tubes, hot-water piping, and other metallic parts of house plumbing and boiler plants. Galvanized steel pipe is attacked by water containing chlorine ingredients, which causes galvanic action between the iron and zinc. Water that contains 500 or more parts per million of sodium or magnesium chloride is dangerous to steel pipe and fittings, especially when heated. Carbonic acid in water is believed to be the chief cause of the formation of tubercles in cast-iron pipe.

Iron in water is objectionable from almost every point of view. A distinct taste is given by 4 or 5 parts of iron per million of water, and less than this amount in laundry water will discolor clothes and cause a reddish-brown sediment when left standing in the air.

Salt gives an objectionable taste to water when more than 250 parts of chlorine per million are present. Alkali salts cause foaming or "priming" in boilers.

Gases seldom are objectionable in water, except as they combine with other substances. A trace of carbonic acid gas seems to render water more palatable. Many of the hydrogen compounds give offensive odors and tastes to water, hydrogen sulphide being probably the most objectionable.

Turbidity causes deposits in reservoirs, pipes, boilers, and

wherever the water comes to comparative rest. It renders water less acceptable for drinking and objectionable for laundry purposes.

Several of the common microscopic organisms cause objectionable odors in water, and sometimes color. They may be so numerous as to give it a turbid appearance and create sediment. When they die in great numbers in a reservoir, their odors may cause a local nuisance. Crenothrix frequents ground water rich in iron and causes a precipitation of iron in solution in water, even choking tubular wells and service pipes. Broken filaments scattered through the water in mains cause deposits of iron rust, making the water unfit for laundry use. Crenothrix thrives best in water containing little or no oxygen but abundance of carbonic acid.

Algæ often grow upon the surfaces of filters, clogging them; and crenothrix grows in the body of a filter treating water high in iron. But the odors given by algæ are the most serious effects commonly experienced. Asterionella give a fishy or aromatic odor; anabæna, a grassy, green-corn or nasturtium odor; uroglena, a fishy and oily odor. These are the algæ which have been most frequently reported as nuisances in reservoirs, but Whipple gives a list of more than a score of offenders ("Microscopy of Drinking Water").

Protozoa, spongiana, rotifera, entomostraca, and certain mollusks are found in reservoir water and even in water mains. Of these, the animals live on vegetable matter and other animals, the vegetable organisms upon soluble inorganic substances and gases (although bacteria seem to be able to feed upon organic matter also). Both thus assist in purifying the water of both organic and mineral impurities. Water mains often contain unsuspected coatings of vegetable and animal life, sometimes called "pipe moss." Fresh-water sponges attach themselves to pipes, the coating often reaching the thickness of a quarter of an inch, and among them may be found mollusks, snails, and many other animal growths. A score or more of different forms have been found in Boston's water mains.

Art. 11. Disease and Drinking Water

It is the physiological effects, and especially those causing serious diseases, that are considered most important in a public water supply. Such a supply is used as drinking water by practically all the inhabitants of a city, and any pathogenic matter in it will thus endanger the entire population.

Most of the inorganic impurities, unless found in large quantities, seem to affect only those not accustomed to them. In those accustomed to soft water, bowel complaints may be caused by drinking hard water, but the same result may follow changing from hard to soft. The same seems to be true of turbidity, water having as high as 20,000 parts per million having been used with apparent impunity in some cities. Organic impurities, also, are believed seldom to cause sickness more serious than temporary bowel complaints, unless present in such quantities as to make the water repulsive. Immunity from all these seems to be acquired by continued use, except from the metallic poisons (lead is about the only one found in water supplies) and pathogenic bacteria; and partial immunity from these also may apparently be acquired.

Typhoid fever and cholera are the two diseases positively known to be caused by bacteria that may be taken into the intestines in water, although diarrhœa and other intestinal diseases are believed to be so caused. Cholera, fortunately, has not been introduced into America; but typhoid (caused by the bacillus typhosus) kills about 20,000 annually in the United States, and at least ten times that number suffer from it. Water is not the sole cause of all these cases, but is believed to be responsible for at least two-thirds of them. The typhoid germ is excreted by those suffering from the disease, and may reach streams either through sewers or by being washed over the surface by rains, and from the streams pass to the water supply of cities below. This bacillus has been known to live for ten days or two weeks in river water, in which time it may travel 500 miles or more. Excrement which has lain frozen on the ground for weeks and been washed into a reservoir by

a spring rain has proved to contain sufficient active bacilli to cause an epidemic. Therefore any stream which receives excreta at any point above where a water supply is taken is dangerous for such use unless first so treated as to remove or kill the pathogenic bacteria.

That the disease is caused by the water supply and that a polluted supply can be rendered innocuous by proper treatment have been abundantly proved by experience. Scores of cities have filtered their water supplies, and almost invariably the typhoid rate has fallen. The following figures give the rates, per 100,000 of population, of deaths from typhoid in a number of cities for five years immediately preceding and five immediately succeeding the introduction of filtration of the water supply, omitting in each case the year in which the change was made.

Albany, N. Y....	170	102	88	100	87	21	32	20	18	19
Paterson, N. J ..	33	46	23	23	34	7	14	4	11	10
Pittsburgh, Pa .	136	139	108	141	135	13	12	10	6	10
Lawrence, Mass...	127	134	119	105	80	19	16	14	34	18

Comparing the average of the five years previous with the average of the five years following filtration, we find the typhoid death rates reduced by the following percentages: Albany, 74; Charleston, S. C., 41; Cincinnati, O., 80; Columbus, O., 78; Harrisburg, Pa., 54; Hoboken, N. J., 28; Indianapolis, Ind., 39; Lawrence, Mass., 79; Louisville, Ky., 58; New Haven, Conn., 38; New Orleans, La., 33; Paterson, N. J., 69; Pittsburgh, Pa., 85; Scranton, Pa., 60; Washington, D. C., 43.

Volumes have been devoted to proving the desirability of preventing these thousands of deaths, since it has been proved that they *can* be prevented. The prevention, if effected by means of a filtration plant, will cost about 35 to 50 cents per year for each consumer, or $35,000 to $50,000 for each 100,000. If this expenditure will prevent 50 deaths per 100,000 population, this makes the cost of saving one life $700 to $1000. But for each death there are at least 10 cases, each costing

at least \$100 for medical treatment and loss of wages, or \$1000 for each death that occurs. The estimated value to the community of the average citizen is given as \$1500 up to five years of age, increasing to \$7500 between 25 and 30 years, and decreasing to \$1000 at 65 to 70 years. The majority of deaths from typhoid occur between the ages of 15 and 35, and the average loss from typhoid deaths is calculated to be \$4635. If filtration costs \$50,000 per year for 100,000 population, the expenditure would pay from a purely financial point of view if the lives saved $= \frac{50,000}{4635+1000} = 9$ per 100,000 population.

Typhoid bacteria are carried to victims by flies, milk, etc., and the use of sterile water would not entirely eliminate the disease. But students of statistics conclude that the "residual" rate can be taken as 7 in the Northern states and 10 in the Southern. Therefore filtration would pay, on the basis of the above figures, if the present typhoid rate equals or exceeds 16 per 100,000 in a Northern city, or 19 in a Southern one.

Art. 12. Investigating Quality of a Water

To determine whether a water is suitable to use for drinking or for commercial or other purposes, and to determine what purification or modification (if any) is necessary to render it suitable, the water itself should be examined to learn what impurities are present, and its source and subsequent history should be investigated to learn the probable origin and significance of these impurities.

The examination should include physical, microscopical, chemical, and bacterial tests. The tests most commonly made are named and classified by Whipple as follows:

Physical Examination: Temperature—turbidity—color—odor (both cold and hot).

Microscopical Examination: Quantity of microscopic organisms per cubic centimeter—amount of inorganic matter, amorphous matter, etc.

Bacterial Examination: Number of bacteria per cubic centimeter—presence of B. coli and other intestinal bacteria associated with pollution.

Chemical Examination: Total residue on evaporation—loss on ignition—fixed solids—alkalinity—hardness—incrustants—iron—chlorine—nitrogen as albuminoid ammonia—nitrogen as free ammonia—nitrogen as nitrites—nitrogen as nitrates—total organic nitrogen (Kjeldahl method)—oxygen consumed—dissolved oxygen—free carbonic acid. (Some of the above are of use in special cases only.)

Quantitative as well as qualitative tests must be made, for in most cases the *amount* of impurity is as important as the *nature* of it. For instance, normal waters often contain a measurable amount of chlorine which is perfectly harmless; but any great excess over the normal is a probable indication of sewage pollution.

Physical examinations are made largely for determining whether the water is palatable and generally favorable for use as a beverage, or whether the turbidity is such as to render it unsuitable for boiler or commercial uses.

Microscopical examinations are of service chiefly to assist in interpreting the other examinations. If the chemical examination leads to a suspicion of sewage pollution, the discovery by the microscope of starch grains, fibers of paper and of muscle, threads of silk, woolen, cotton, or linen, would tend to confirm such suspicion. It is in the detection and identification of living organisms, however, that microscopical examination is most valuable. Chemical tests do not determine directly whether the organic matter present in a water is animal or vegetable, living or dead; but, if it is living, the microscope reveals this and generally the species as well. Such an examination may serve to explain the cause of odor and taste; whether high albuminoid content is due to sewage or other pollution with organic refuse matter, or to the presence of some organisms such as algæ in sufficient quantities to be revealed by the chemical test; what organisms are attached to the walls of water pipes, what are growing upon the surface of a filter bed, etc.

Bacterial examinations in connection with water supply are employed almost exclusively for detecting the presence and determining the number of bacteria that are indicative of sewage pollution. Since sewage contains more bacteria than any other liquid that ordinarily finds its way to streams, high

numbers in a water are assumed to indicate sewage pollution. They do not prove it, however, since this abundance *may* be due to another cause. The number of B. typhosus in even sewage itself is so relatively small that a bacteriologist might spend a lifetime examining polluted water without ever finding an individual of this species.

Chemistry enables us to determine both quantitatively and qualitatively the presence of the elements, and in many cases we can determine also in what combinations they exist, and in still other cases can infer such combinations. By chemical tests we can determine how much temporary or permanent hardness a water has, how much iron it carries, how much lead, how much salt, or other mineral substances. The chemical test is the principal one used for determining whether a water is suitable for commercial uses.

By chemical tests we can determine the presence of organic matter, but not whether it contains life. Certain conditions or combinations, however, have been found to be due to the action of living organisms, and chemistry may, by showing the existence of such conditions, enable us to infer the presence of such living organisms.

As an illustration of the use of the several tests, the chemist may determine that a given river water contains more than a normal amount of nitrogen, and, knowing that sewage contains a large amount of nitrogen, may suspect sewage contamination. But he cannot be sure that the nitrogen was not washed out of the air by rain or from peaty deposits by ground water. The bacteriologist examining the same water may find it high in bacteria, including B. coli, a species seldom (but sometimes) found outside of sewage, and this tends to confirm the suspicion of sewage pollution. The microscopist may find in this water filaments of cotton, woolen, etc., which probably form parts of matters discarded by human beings; and the odor of the heated water may be characteristic of sewage. All these form cumulative circumstantial evidence, so to speak, but no one of them warrants a decided opinion, and all combined furnish an opinion only and not a proof.

In a determination of sanitary qualities, "The value of an analysis lies in its interpretation, and each part of the analysis must be interpreted by comparison with all of the others and in the light of exact knowledge of the environment of the water. The interpretation of an analysis is as much a matter of expert skill as is the making of the analysis itself. The physical, biological, and chemical examinations should be interlocking in their testimony, yet these different parts are to be given different weight in the study of different problems. For example, in the detection of pollution, the chemical and bacterial examinations furnish the most information; in the study of the æsthetic qualities of a water, the physical and microscopical examinations are most important; while in investigations concerning the value of a water for industrial purposes, the chemical and physical examinations alone may suffice." (Whipple, "The Microscopy of Drinking Water.")

Note that the above refers to a sanitary analysis. For determining mineral constituents only, chemical analyses are ordinarily accurate and reliable.

Sanitary Survey. Reference is made in the above quotation to "exact knowledge of the environment of the water." If the object of the analysis referred to in the illustration given was to determine whether the water contained sewage pollution, and if a sewer was known to discharge into the river at a point a mile or less above where the sample was taken, such knowledge furnishes more absolute proof than any possible number of analyses could; although the analyses could show what chemical effect such sewage had had upon the water, how it had affected its physical appearance and to what extent it had increased the number of bacteria present.

This knowledge of environment is obtained by making a "sanitary survey" of a stream or reservoir site. The making of such survey is a very important duty which is too often neglected. Information concerning water pollution obtained in this way is not only final and definite, but it reveals the source of pollution, which must be determined in any event if the pollution is to be prevented or abated. Such a survey has as

its purpose the discovery of any sewers which may be discharging into the source of supply or any of its feeders, of any discharge thereinto from private water closets, pigpens or other places frequented by animals, from industrial plants, or swamps, or of any other sources of pollution. Such a survey should be made at least once a year of every public water supply.

Art. 13. Making and Interpreting Analyses

While it is not necessary and is seldom possible that a waterworks engineer should be an expert chemist or bacteriologist (and the tests and conclusions of one who is not expert are worse than useless), he should understand the principles and aims of the tests that are made by experts, should be familiar with the terms and units employed in reporting such tests, and should have a certain general knowledge of the significance attached to the results obtained.

In stating the amounts of the several impurities in water, certain standard units of measurement must be employed, the meanings of which are defined with great precision.

Units of Measurements. Quantitative statements of living organisms are made in terms of the numbers of individuals present. Quantities of the various elements present are usually expressed in parts per million of the containing water, on the basis of weight. The amount of free oxygen is also sometimes described as a certain percentage of "saturation," that is, of the amount that is contained in pure water under normal conditions when saturated with air.

Properties or qualities of water, of course, cannot be based upon weight units, but others must be employed. The following are those commonly used:

Turbidity, although caused by matter in suspension and varying with the amount of such matter, is measured by a unit based upon appearance rather than quantity. This unit is the distance beneath the surface of the water at which a platinum wire 1 millimeter in diameter can just be detected by eye. To apply this standard, a hardwood rod one-half

inch square and about 4 feet long has such a wire inserted at right angles to it near one end, and an open eye-piece near the other end 47¼ inches (1.2 meters) distant from the wire. The eye is placed at this eye-piece and the other end of the rod lowered gradually into the water until the wire just disappears. This test should be made in the open with a clear sky, but not in direct sunlight. The turbidity is measured by an arbitrary scale, that recommended by the U. S. Geological Survey being in common use. In this scale that turbidity that causes the wire to disappear at a depth of 100 millimeters is called 100, and the other turbidities are assigned numbers as indicated in the accompanying table:

GRADUATION OF TURBIDITY ROD

Turbidity.	Depth of Wire, mm.	Turbidity.	Depth of Wire, mm.
10	794	160	69
15	551	180	62
20	426	200	57
25	350	250	49
30	296	300	43
40	228	350	39
50	187	400	35
60	158	500	31
70	138	600	28
80	122	800	23
90	110	1000	21
100	100	1500	17
120	86	2000	15
140	76	3000	12

The quotient obtained by dividing suspended matter (expressed in parts per million) by turbidity is called the "turbidity coefficient." It is usually between 0.4 and 0.6, becoming greater as the suspended matter becomes coarser.

It is apparent from the above that turbidity is considered from the viewpoint of opacity rather than of quantity, since, in comparing that caused by different kinds or colors of soils or other materials, equal turbidity may be produced by unequal amounts of these materials.

Turbidity of the same stream varies from day to day, being on some days 1000 times as great as on others in certain rivers.

Color is expressed in parts per million of a standard coloring matter required to give corresponding opacity to distilled water. The standard coloring matter consists of 1.246 grams of $PtCl_4 2KCl$, containing 0.5 gram of platinum, and 1 gram of $COCl_2 6H_2O$, containing 0.25 gram of cobalt, dissolved in 100 cubic centimeters of concentrated hydrochloric acid and enough distilled water to make the whole equal 1 liter. This is called a color of 500 parts per million. Standards of any lesser intensities are made by diluting this with distilled water. These standards are placed in Nessler tubes of such dimensions that the 100 cubic-centimeter mark comes about 25 centimeters above the bottom. The water to be tested is placed to the same depth in a similar tube and this and the standard tubes are compared by looking down through them while they stand on a white porcelain plate. The water to be tested is first filtered to remove all turbidity.

The color of a given water may undergo seasonal variations, but these are not so great nor nearly so rapid as the changes in turbidity.

Hardness is expressed in parts per million of alkalinity (note that not all alkalinity is "hardness"), or is determined by the soap test. In the United States, water with 50 or less parts per million of hardness is called "soft"; and that with 100 or more parts per million is called "hard." In England water with less than 150 parts is called "soft." The soap test is only approximate and consists in determining the amount of a standard soap solution that must be mixed with a given amount of the water to produce a lather that persists for five minutes. The result is expressed in "degrees" (each degree corresponding to 1 part of $CaCO_3$ to 70,000), or in parts per million.

The hardness of a given water is practically constant, except when it is due to pollution.

Taste and Odor. There are no standards of taste and odor, but merely terms which have been commonly adopted for

describing individual impressions as to them. Among the terms used are "very faint," "faint," "distinct," "decided," "very strong," "aromatic," "grassy," "nasturtium," "cucumber," "fishy," "pigpen," "chemical."

Taste and odor come and go in most waters, remaining for only a few days at a time.

Bacteria are reported as a certain number per cubic centimeter of water. Since 1 cubic centimeter may at times contain thousands and even millions of bacteria, it is, of course, impossible in such cases to count the total number. If there be reason to suspect the presence of more than 200 per cubic centimeter, 1 cubic centimeter of the sample is mixed thoroughly with 9 cubic centimeters of sterilized or distilled water. If more than 2000 are anticipated, 1 cubic centimeter of this diluted mixture is again diluted with 9 cubic centimeters of sterilized water; and the successive dilutions may be continued as many times as is necessary to reduce the average number of bacteria per cubic centimeter to 200 or less. The number of bacteria found in 1 cubic centimeter of diluted water is then expressed as such number with as many ciphers added as there have been dilutions.

One cubic centimeter of the original water or its dilution is then placed in a sterilized Petri dish and to it is added 10 cubic centimeters of standard agar at a temperature of 40° C., and the agar and water are mixed by tipping the dish back and forth. The mixture is cooled and placed in an incubator for forty-eight hours at 20° C. The assumption is that the bacteria present are distributed throughout the mixture in the dish and that each of them generates numerous other bacteria until the group or "colony," as it is called, is sufficiently large to be visible; each colony being then taken as representing 1 bacterium in the original water. The number of colonies is counted by the aid of a lens.

Another test is frequently made, using litmus-lactose-agar instead of agar and the mixture is incubated at a temperature of 37° C. for twenty-four hours. These being standard tests, the use of the terms 20° and 37° counts serves to distinguish

between them. In the latter case the number of red or acid-forming colonies also is recorded. The bacterial count at 20° indicates water- and soil-bacteria; the count at 37° those of sewage origin; but this distinction is not rigid. The ratio of the 37° count to the 20° count is taken as an index to the pollution of the water. For pure water this is 1 : 10 or more; for very polluted water it may be 1 : 1. Red colonies are considered to indicate B. coli.* Satisfactorily filtered water should give a 20° count below 100, only a few on agar at 37°, and very infrequent red colonies on litmus-lactose-agar.

Another test for B. coli communis is made by use of standard lactose-peptone bile used in test tubes.* Each tube is inoculated with 1 cubic centimeter of each of three or more dilutions, so selected that the probability is that the highest dilution will contain no B. coli. A small test-tube about 3 inches long is inverted and dropped in the larger tube, and the latter is then plugged with cotton. The tubes are then incubated at 37° C. for forty-eight hours, and all tubes showing 20 per cent or more of gas in the small inverted tube are called positive, that is, it is assumed that B. coli were present. The number of those present is not indicated, but some assume that in the highest of the dilutions giving a positive result, only 1 bacterium was present. If, for instance, dilutions of 1 : 10 and 1 : 100 gave positive results, while no gas showed in the 1 : 1000 dilution, it might be assumed that there was 1 B. coli in the 1 : 100 dilution. This is not the common practice, however, but the results are ordinarily indicated by the use of the plus or minus sign to indicate the presence or absence of gas in the different dilutions.

The number of bacteria may increase or decrease in a sample of water after it is taken; therefore the tests of the water should be made immediately, or the samples kept iced.

The methods of making water anlayses have been standardized by the American Public Health Association; and these

* Both of these are presumptive tests for B. coli. There are other bacteria that give the same indications in these tests, although their presence is not so probable as that of B. coli. Other standard tests should be made if greater certainty is desired.

methods are explained in detail in the "Standard Methods of Water Analysis" published by that society.

Samples. Examinations of water are made for public plants by most state boards of health, and there are numerous laboratories that make a specialty of this. In sending samples of water to them, great care must be taken. The bottle in which a sample is collected and sent should be sterilized by boiling in water or steaming, and a glass stopper should be used and similarly sterilized. (The laboratory will generally furnish a bottle already sterilized and sealed.) If a sample is to be taken from a faucet, let the water flow until that standing in the service-pipe has wasted. Open the bottle while the water is running and at once fill the bottle, rinse it and fill again; rinse the stopper and insert it while the water is running over it. In the meantime, do not let anything but the water come in contact with stopper or neck of bottle. If the sample is taken from a river or other body of water, open the bottle under the water, rinse it and refill and insert the stopper while it is still under the surface of the water. At once cover the stopper with a cover of tin-foil or cloth, tie this firmly in place and seal it. Then send it to the laboratory by the quickest agency. For bacterial analyses it is desirable that the bottle be packed in ice in transit. The bottle should contain not less than a half gallon, and gallon bottles are commonly used for this purpose.

Significance. Most minerals found in water have little or no significance except as to their direct effect upon the acceptability of the water. To illustrate: calcium carbonate, if found in water in small quantities which do not lessen its value for boiler, household, or other uses, may be disregarded. But there are some minerals that have a special significance because of the fact that they may indicate pollution.

Organic matters found in water must be studied to learn whether they are due to leaves or other harmless matters, or to pollution by sewage or other animal filth.

Chlorine. Of the minerals, chlorine is the most important in its indirect significance. It is found in normal water as

far west as Ohio, being carried inland as finely divided sea-spray. In certain relatively small areas of this eastern section, as around Syracuse, N. Y., there are natural deposits of salt in the earth which cause the streams to show considerable chlorine content, and west of Ohio a large proportion of the natural waters are more or less affected by salt deposits in the soil. The amount of chlorine derived from ocean spray varies quite uniformly with the distance from the ocean. In several of the states the normal amount in unpolluted water has been determined for a great many localities and maps have been prepared showing "isochlors," i.e., lines so located that, at every point on a given "isochlor," unpolluted water contains the same amount of normal chlorine. Fig. 1 shows the isochlors for New England and New York.

Aside from these two sources, almost the only other source of chlorine is domestic drainage. Salt always occurs in drainage from animal sources, because in all animal economy a certain fairly definite amount of salt is taken in with the food and the same amount is expelled from the body, still as salt; and the salt persists after all organic matter has been removed from the water by chemical or bacterial action, by sedimentation or filtration. Chlorine more than normal in amount is, therefore, a strong indication of sewage pollution, although it is impossible to tell how far distant in either space or time such pollution occurred. Certain trade wastes discharged into streams carry salt, and much is contributed to some streams by flow from oil well borings, and chlorine so contributed of course has no bearing upon the determination of sewage pollution.

Organic matter in water appears as living organisms, animal or vegetable; products of organic life, as albumen, urea, tissue, etc., dissolved or suspended; and products of decomposition of organic matter, such as salts of ammonia and carbonic and nitric acids. Carbon and nitrogen oscillate between the organic and the inorganic state. Organic matters cannot be determined directly by chemical analyses, but only by indirect methods; the nitrogen compounds being generally taken as

an index of the amount present, since "it is the nitrogenous organic matter which has the greatest sanitary importance, owing not only to the facility with which it undergoes decomposition, but also to the fact that nitrogen is an essential element in all living matter. Analytical processes of great accuracy enable us to determine nitrogen in four forms; namely, as organic nitrogen ('albuminoid ammonia'), as ammonia, as nitrous acid, and as nitric acid." (Mass. State Board of Health.)

Organic matter consists chiefly of carbon, hydrogen, nitrogen, and oxygen. When its life departs it begins a decomposition, first by oxidation of the carbon, which leaves the nitrogen combined with hydrogen in the form of ammonia; and subsequently by the oxidation of the ammonia to nitric acid (NH_3 to HNO_3), which generally combines with some mineral base in the water. The ammonia (free ammonia) discovered by chemical analysis indicates that organic matter once present has begun rapid decomposition. That not yet decomposed, whether living or dead, is changed by the addition of chemicals to ammonia and reported by the chemist as "albuminoid ammonia"; while ammonia that has been oxidized is recorded as "nitrites" or "nitrates," according as nitrous or nitric acid has been formed. Albuminoid ammonia is about $\frac{14}{17}$ nitrogen, and the amount of this obtained by the analysis is about one-half the organic nitrogen present. About $\frac{4}{25}$ of animal matter and a much smaller part of vegetable matter are nitrogen; algæ containing about $\frac{1}{15}$ nitrogen. Hence the albuminoid ammonia, if derived wholly from algæ, times $\frac{14}{17} \times 2 \times 15$ would give approximately the amount of algæ present. It is now common practice to determine also the "total organic nitrogen" present by what is known as the Kjeldahl method.

The absorption of oxygen in the formation of nitrous and nitric acid generally uses up much of the free oxygen in the water, and hence the absence of oxygen in the water is sometimes considered as an index of the organic matter present. The amount of oxygen absorbed from potassium permanganate or other oxidizing agent by the water to replace that used

up is generally stated as "oxygen consumed." When saturated with oxygen, water at 32° Fahr. contains 14.7 parts of this by weight per 1,000,000; and at 80°, 8.1 parts. The amount of "dissolved oxygen" is generally determined in parts per million.

The oxidation of the organic matter takes time, and also requires the presence of free oxygen in the water; the greater the amount of oxygen, the more rapid the oxidation, generally speaking. If there is not sufficient free oxygen, the oxidation will not take place. Apparently nitrates may yield part of their oxygen to unoxidized matter and by so doing revert to the nitrite form.

High ammonia, nitrites, and chlorine together form an almost sure indication of sewage pollution. The excreta from each person have been found to contribute daily to sewage an average of .015 pound of free ammonia, .003 pound of albuminoid ammonia, .218 pound of dissolved solids, and .042 pound of chlorine. These amounts will, of course, vary somewhat with the age, sex, and food-matter of each contributor; but by their use an approximate idea can be formed of the pollution of a stream of given flow which is caused by a given number of sewage contributors.

ART. 14. QUALITIES DESIRABLE IN A WATER SUPPLY

On a knowledge of the principles and considerations outlined above we can base a judgment as to what qualities are desirable in water furnished as a public supply. These are as follows:

Matter in Suspension. Any matter in suspension soils clothing in private and public laundries; causes deposits in pipes, reservoirs, boilers in kitchens, and in steam plants; and its presence in drinking water is repulsive to most persons. In addition, if the suspended matter be organic, it is liable to undergo putrefactive decomposition, especially when deposited in pipes or reservoirs, and impart offensive odors to the water, and in some cases color as well; also sulphuric and other corrosive acids; and in this form it is undoubtedly

productive of intestinal trouble in those who take it with their drinking-water.

Water should, therefore, be entirely free of suspended matter, or as nearly so as possible.

Bacteria are, of course, suspended matter, but are so minute and constitute such a small part of the water even when present in enormous numbers, that they do not form a sediment and are not visible. Their objectionable characteristic is the morbific result of their life processes. Of millions present in a glassful of water, the probabilities are that only one or two are pathogenic, and possibly none is. But the possibilities of the presence of any disease-producing bacteria are so serious that every effort should be made to reduce to a minimum the number present in drinking-water. (If the water is never used for drinking, the presence of bacteria is of no importance.) Since considerable numbers of bacteria imply large amounts of organic matter, and since such amounts are commonly due to sewage; and since pathogenic bacteria may occur at any time in sewage-polluted water, therefore, the presence of large numbers of bacteria in a water should be considered suspicious and the cause thoroughly investigated.

The efficiency of a purification plant is frequently expressed in percentage of bacteria (or sometimes other impurity) removed. This is not a reliable test of either the efficiency of the plant or the safety or acceptability of the effluent. For if a water contain a million bacteria per cubic centimeter, a removal of 99.5 per cent would still leave 5000 bacteria in the effluent; while if the raw water contained only 200 bacteria, the same percentage of removal would leave but 1 in the effluent. Almost any filter could perform the former, almost none the latter. And the former effluent would still be of questionable safety. The better plan is to specify the average and maximum numbers allowable or obtained in the effluent.

Matters in Solution. Chemically pure water is not obtainable or desirable. A certain amount of hardness (some say 10 parts per million) renders it more palatable and probably plays an important part in the physiological functions, and the same

is true of chlorine (salt) up to 2 or 3 parts. Possibly minute portions of other minerals are beneficial as well, but of this little is known.

Lead is poisonous in large doses, or taken continually for a long time in small doses (since each increment of lead remains in the system for some time), and should not be contained in drinking-water. Its only probable source is lead pipes.

Sulphur is objectionable because it will probably be or become in the form of sulphuric acid, which is corrosive of metal pipes and plumbing fixtures; beause it may occasion most offensive odors (sulphuretted hydrogen); and because it may be poisonous to the human system.

Iron is objectionable in any amount because it is liable to pass into the insoluble form by the absorption of oxygen, and stain clothing, wash-basins and everything else it comes in contact with; in addition to which there are the general objections to suspended matter.

The latest standard of purity to be accepted by water specialists is that prepared in 1914 for the U. S. Treasury Department by a committee of 15 experts, and required by said department to be observed by all inter-state common carriers in connection with water supplied by them for drinking. This standard has been adopted by a number of cities and other bodies. The standard is a bacteriological one only, and is as follows: *

The following are the maximum limits of permissible bacteriological impurity:

1. The total number of bacteria developing on standard agar plates, incubated twenty-four hours at 37° C., shall not exceed 100 per cubic centimeter. Provided, that the estimate shall be made from not less than two plates, showing such numbers and distribution of colonies as to indicate that the estimate is reliable and accurate.

2. Not more than one out of five 10-cubic-centimeter portions of any sample examined shall show the presence of organisms of the bacillus coli group when tested as follows:

*Reprint from the *Public Health Reports*, Nov. 6, 1914. A committee was appointed in 1922 to revise this standard but has not yet (January, 1924) presented a final report.

(*a*) Five 10-cubic-centimeter portions of each sample tested shall be planted, each in a fermentation-tube containing not less than 30 cubic centimeters of lactose peptone broth. These shall be incubated forty-eight hours at 37° C. and observed to note gas formation.

(*b*) From each tube showing gas, more than 5 per cent of the closed arm of fermentation-tube, plates shall be made after forty-eight hours' incubation, upon lactose litmus agar or Endo's medium.

(*c*) When plate colonies resembling B. coli develop upon either of these plate media within twenty-four hours, a well-isolated characteristic colony shall be fished and transplanted into a lactose-broth fermentation-tube, which shall be incubated at 37° C. for forty-eight hours.

For the purposes of enforcing any regulations which may be based upon these recommendations the following may be considered sufficient evidence of the presence of organisms of the bacillus coli group:

Formation of gas in fermentation-tube containing original sample of water (*a*).

Development of acid-forming colonies on lactose-litmus-agar plates or bright red colonies on Endo's medium plates, when plates are prepared as directed above under (*b*).

The formation of gas, occupying 10 per cent or more of closed arm of fermentation-tube, in lactose-peptone broth fermentation-tube inoculated with colony fished from twenty-four hour lactose-litmus-agar or Endo's medium plate.

These steps are selected with reference to demonstrating the presence on the samples examined of aerobic lactose-fermenting organisms.

3. It is recommended, as a routine procedure, that in addition to five 10-cubic-centimeter portions, one 1-cubic-centimeter portion and one 0.1-cubic-centimeter portion of each sample examined be planted in a lactose-peptone broth fermentation-tube, in order to demonstrate more fully the extent of pollution in grossly polluted samples.

4. It is recommended that in the above-designated tests

the culture media and methods used shall be in accordance with the specifications of the committee on standard methods of water analysis of the American Public Health Association, as set forth in "Standard Methods of Water Analysis" (A. P. H. A., 1912).

CHAPTER IV

PURIFICATION OF WATER

Art. 15. Methods of Purification

By purifying water is meant totally or partially removing impurities from it, or else so changing them that their presence is less objectionable. An illustration of the former is the removal of suspended clay by sedimentation or filtering; an illustration of the latter is the bleaching of coloring matter or the destruction of bacteria by sterilizing agents.

For removing suspended matter the processes are sedimentation, in some cases assisted by coagulation; slow sand filtration, in some instances assisted by coagulation; rapid sand filtration, always assisted by coagulation; prefiltration—rapid filtration without the aid of coagulation.

Matter in solution can be removed only by first rendering it insoluble by changing its chemical combination, and then treating it as suspended matter. In some cases combinations are effected which result in the water taking into solution new mineral matters which are much less objectionable than the old.

Living organisms may be removed as suspended matter. They may also be deprived of life by applying substances known to be poisonous to them (the treatment of bacteria in this way is called "disinfection" or "sterilizing"), or by subjecting them to unfavorable environment such as removing their food supply.

In most cases at least two and sometimes four or five different processes are used in treating a water. Effectiveness and economy should both be considered in selecting the combination of processes, with careful regard to their mutual interrelation and most favorable sequence of use. The general conditions under which each is most suitable are named below; but it should be clearly understood that various local conditions or

peculiar characteristics of the water may call for modifications of this general practice.

Sedimentation is the cheapest and generally the best method of removing the coarser suspended matters. The finer the matter the longer it takes it to settle, and consequently the larger must be the basin provided for holding the water while sedimentation takes place. If the water must pass through a large reservoir anyhow, clarification (that is, removing turbidity) may well be continued until practically all of the suspended matter has settled out. It is cheaper to remove the sediment in a mass from a reservoir than to remove it by a filter, provided the size and cost of the reservoir do not enter in as a factor.

Coagulation is a process by which there is introduced throughout a water a hydrate of aluminum or of iron or other coagulant, the peculiar property of which is that it collects into flocculent masses very fine suspended matter, which in this form settles much more rapidly than it would otherwise, the matter so collected including colloids and bacteria which of themselves would not settle at all. Coagulation also removes many kinds of matter in solution. This collection of suspended matter into masses also permits straining out these fine particles in filters through which most of them would individually pass unimpeded. The coagulant is but little heavier than water, and settles very slowly; and it may therefore be economical to remove it and the enmeshed suspended matter by filtering rather than by the long storage required for sedimentation.

Since sand and other coarser matters will settle rapidly without coagulation, it is not necessary to waste expensive coagulant on them; therefore in many plants the coagulant is not added to the water until after the coarser matters have settled out.

If a basin must be built expressly for sedimentation, it is a matter of nice calculation and adjustment between this and the succeeding treatment to determine what size of basin will be most economical. This will depend largely upon the characteristics of the suspended matter, but also upon the kind of filter or other process used for completing the clarification.

In the treatment of any given water, and using a given kind of filter, there will generally be a point beyond which further removal of suspended matter can be accomplished more economically by filtration than by sedimentation.

Rapid filtration (also called mechanical or American filtration) consists in passing water that has been coagulated through a bed of sand at a rate of about 100 million to 125 million gallons per acre per day.* The coagulated matter collects on or near the surface of the sand bed. If the coagulation has been properly performed, the filter will retain practically all of the suspended matters, not more than 0.5 of 1 per cent of the bacteria coming through with the effluent. The matter so collected in the filter is removed by washing the sand about once a day. (The frequency varies with the rate at which coagulated matter collects in the filter.) If the fine suspended matter were not coagulated, most of it would pass through the filter. It is by the use of such a filter that coagulated matter that has not settled out in the sedimentation basin is removed. But if there is any considerable amount of suspended matter in the raw water, applying all of this to the filter without previous sedimentation would require frequent and expensive washing of the sand and a large filter plant, which would in many cases be less economical than removing a part of it by sedimentation, as stated above.

A slow sand filter (also called an English filter) is a bed of sand of finer grains than are used in a rapid filter, and through which uncoagulated water is passed at a rate of about 3 million gallons a day. (In a few cases coagulation is used occasionally, but these are exceptional.) This rate is equivalent to $4\frac{1}{2}$ inches of vertical movement per hour, which is so slow that part of the suspended matter is deposited in the top layer of sand. Also there forms here a mat of bacterial jelly (called by the Germans Schmutzdecke), which also coats each particle of sand to a considerable depth and attracts and holds bacteria and other fine suspended matters. The bacteria that develop in

* 175 million gallons were filtered successfully at Detroit in 1918, this water carrying only a small amount of suspended matter.

the bed bring about an oxidation of the organic matter and sometimes other minor changes. The accumulated matter is removed from these filters by shoveling off a thin top layer of sand. (Exceptions to this will be described later on.) As slow sand filters are about 40 times as large as rapid filters, this cleaning is expensive; and if coagulants were used, such frequent cleaning would be required as to make the cost prohibitive. It is therefore evident that these filters are not well suited to treating coagulated water. Nor can they be used successfully for water carrying much turbidity.

Comparing the results obtained with slow and rapid filters, it may be said in a general way that thev are equally effective in removing bacteria; the slow filters are much less effective in removing fine suspended matter and color, but much more so in the oxidizing of organic matters.

If the water contains iron, that in solution can be thrown out of solution by aeration; if it be hard, the alkalies can be thrown out of solution. In either case the matter so rendered insoluble is removed by filtration, frequently preceded by sedimentation.

Waters carrying other mineral matters in unusual amounts require special chemical and mechanical treatments; but these are exceptional cases, each of which must be treated as a separate problem.

Either slow or rapid filters will *remove* 99 to 99.7 per cent of all the bacteria present. There are several methods of treatment in common use for *destroying* bacteria, but which do not remove them or any other matters. These are known as "disinfection" or "sterilization." The methods commonly employed use chlorine as the agent; but ozone and violet rays have been successful experimentally.

Algæ in reservoir water may be killed by poisoning them, copper sulphate being the agent which is believed to be best for this purpose, everything considered. For removing them from water after it has left the reservoir, filtration or some variation of it is generally employed. For removing odors and certain other objectionable qualities or matters, abundance of

oxygen is supplied by various methods of aerating, generally by causing the water to fall through the air in sprays or sheets. Such treatment also releases gases entrained or in solution in the water.

The above are but general statements, given briefly for the purpose of showing what methods are available and for what purpose each is specially adapted. The several methods will be discussed at length in the following pages. From this brief review, however, we may draw some general conclusions as to the methods most effective for treating the various combinations of impurities in order to render a water safe and acceptable for a public supply.

If a water is fairly clear, but sewage contamination is known or suspected, the slow sand filter is probably best. (Good practice requires treating for pathogenic bacteria all water that flows for any distance over the ground surface or in open streams, since these may at any time be polluted.) If it contain always or at frequent intervals suspended matter, most of which will settle rapidly, slow sand filters preceded by sedimentation can be used. If considerable very fine matter is frequently present, sedimentation and rapid sand filtration are probably best. If there is not much fine matter, but the water carries considerable color which it is desired to remove, the rapid filter may be best. If the water is treated for removal of iron or hardness by throwing them out of solution, again the rapid filter is preferable for removing the matter rendered insoluble. Iron is rendered insoluble by aerating; hardness alkalies by the addition of certain mineral matters. If odors are to be removed, aerating is generally employed, or a slow sand filter (in which filters oxidation takes place), or frequently both. If a water is satisfactory in every other respect, contains no appreciable suspended matter, but may possibly at times carry pathogenic bacteria, then disinfection alone may be employed. It is becoming quite common practice to provide disinfecting apparatus and materials in all purification plants, to be used when the water is unusually polluted with sewage, or in case of a temporary decrease in efficiency of the

filtration plant. Scores of plants now use it regularly as a final process for destroying practically all of the bacteria that may escape the filtration or other processes.

Art. 16. Sedimentation

Although sedimentation is the oldest and simplest method of purifying water, the natural laws controlling it are but little understood, and most sedimentation basins are designed largely by guesswork or rule-of-thumb. Almost any basin will effect some sedimentation, but the amount may often be increased several fold by using, in designing and operating, the knowledge that is available. The following presents briefly the ideas of leading engineers who have given the subject thought and investigation.

Suspended matter settles through water owing to gravity, but the motion is resisted by friction between the surface of each particle and the water, and also by the viscosity of the water. The viscosity resistance varies with the temperature, being about twice as great at 32° F. as at 74° F. For large particles the resistance due to friction is much greater than that due to viscosity, but as the particles decrease in size, the relative effect of viscosity increases. Friction resistance varies as the square root of the diameter, viscosity as the square of the diameter. According to Hazen, the settling velocity of particles larger than 1.0 millimeter diameter is controlled chiefly by friction, that of those less than 0.1 millimeter diameter is controlled largely by viscosity, and between those diameters the velocity varies approximately as the diameters. He gives the settlement velocities of particles of mineral sediment below 1.0 millimeter diameter, at 50° F., as shown in Table No. 7.

The last rate is equivalent to 1 foot in 6000 hours, or about $1\frac{1}{2}$ feet per year—a rate of no practical service whatever. Many bacteria and some clay particles are smaller even than .0001 millimeter.

No water in reservoirs or basins is absolutely quiet. The friction of moving water within itself is very slight, and the

least inequality of pressures causes motion, and motion once imparted persists for a long time through inertia. Very slight changes in temperature, which change the specific gravity of the water, or the friction or pressure of wind blowing over the surface may cause motion; while the velocity of the water as it enters the basin causes motion in the entire body of water which cannot be entirely prevented.

Table No. 7

VELOCITIES AT WHICH PARTICLES OF SEDIMENT FALL IN STILL WATER AT 50° F.

Diameter of Particles, in Millimeters	Settling Velocity, in Millimeters per Second.	Diameter of Particles, in Millimeters	Settling Velocity, in Millimeters per Second.	Diameter of Particles, in Millimeters.	Settling Velocity, in Millimeters per Second.
1.00	100	0 08	6	0 006	0 055
.80	83	.06	3 8	005	0385
60	63	05	2 9	004	0247
50	53	04	2 1	003	0138
40	42	03	1 3	002	0062
30	32	02	0 62	0015	0035
.20	21	015	0 35	001	00154
.15	15	010	0 154	0001	0000154
10	8	008	0 098		

These motions, and especially those caused by temperature changes and wind pressures, cause vertical currents which tend to keep matter in suspension and retard sedimentation. It is also believed that any longitudinal motion along the bottom of a basin (or the top of sediment collected in the bottom) causes sufficient upward current or pressure to keep from final settlement particles whose settling velocity is less than the longitudinal velocity of the water.

It has been shown by numerous experiments that after the coarser matter—probably that larger than 0.1 millimeter—has settled out, the remainder continues distributed uniformly throughout the depth of the basin, although it gradually settles out. That is, at any time previous to complete clarification

there is as much of the matter then in suspension in the top stratum of water as in that just above the settled matter; except that some believe that the top inch or two of water clears more quickly than the rest. The reason for this is not definitely known.

If water were perfectly quiet in a basin 2 meters deep and the velocities in Table No. 7 are correct, particles 1.0 millimeter in diameter would reach the bottom in twenty seconds. During the same time particles 0.5 millimeter in diameter would have settled from a distance of 1.06 meters above the bottom, and 0.1 millimeter particles would have settled from a bottom stratum 0.16 meter deep, etc. That is, the bottom layer of sediment would contain all the 1.0 millimeter matter, half of the 0.5 millimeter matter, and about one-sixth of the 0.1 millimeter matter, and the other sizes in proportion.

If, instead of being quiet, the water moved as a mass slowly from one end of the basin to the other, the particles would settle at the same rate as before, but would at the same time have a longitudinal movement. If the longitudinal movement continues long enough to permit the particle to reach the bottom before the water leaves the basin, it will remain in the basin; otherwise it will probably be carried out with the water. As stated, the longitudinal velocity of the bottom particles of water probably must be less than the settling velocity of the particles of suspended matter to be retained. Little is known of the relative velocity of these bottom particles compared to the mean velocity of the water as it flows through a basin, but it possibly will average about one-third of this. If this ratio be assumed, then the mean velocity of flow through a tank may be three times the settling velocity of a given size of particle without preventing the settling out of such particles.

From this it would follow that there would be little advantage in having a tank longer than that required to give the smallest particles to be removed (which we will call the "limiting" particles) time to reach the bottom. As a matter of fact, the first part of a tank contains currents and vortexes caused by the velocity of the entering water; and near the outlet the velocity

increases as the water concentrates to the smaller area of the outlet stream; so that in a basin of ordinarily good design, possibly only one-third of the length is acting to best advantage, and the full length of the basin should be three times that just given.

A basin which water neither enters nor is drawn from during sedimentation is called an "intermittent basin." One through which the water flows during sedimentation is called a "continuous flow basin." In using the former there are three periods: filling, settling, and emptying. If the filling is done too rapidly, currents and vortexes will be set up which will continue for hours or even days. If the water is withdrawn too rapidly it will stir up and remove matter that has settled out. If the three periods are of equal length, then three basins must be provided, each capable of holding an amount equal to the average quantity clarified by the entire plant during one period; and a fourth basin for use while one of the others is being cleared of sediment. Since the water must all be drawn off, the outlet must be several feet lower than the inlet—in many cases a serious objection.

A continuous-flow basin empties at a level only an inch or less lower than the inlet. The velocity of flow of the entering water may generally be less than in the case of the intermittent basin. And if the currents are properly controlled by baffles or otherwise, a continuous-flow basin will give as good sedimentation as a triple intermittent basin, or even better. Most modern basins are therefore operated as continuous flow.

It is seen from the above considerations that the size of the limiting particles may be determined theoretically by the proportions of the basin. A basin of a given depth and length will remove practically all of the particles larger than a given size; or, having decided upon a limiting size, the proportions of the basin may be determined. Unfortunately, there is no definite knowledge as to what allowance must be made for uncontrollable currents in the basin, nor even the best way to minimize this effect.

The assumption that the mean velocity of flow through a

tank must not exceed three times the settling velocity of the limiting particles gives a method for determining the vertical cross-section of a basin, the area of this equaling the quantity of flow per second divided by three times this settling velocity. This leaves the relative depth and width undecided. Since the length varies directly as the depth, the horizontal area of the tank is a constant, regardless of the depth, and consequently a shallow tank would seem to be best because cheapest. But it is generally thought that the tank should be at least 5 to 8 feet deep, to prevent a stirring up of sediment by surface currents. Tanks as deep as 50 feet have been built, but 20 feet seems to be a more desirable limit of depth. Probably the wisest plan is to use that depth greater than 5 feet that will prove most economical for construction. It should be noted that the calculated depth should be available at the end of a run, when the greatest amount of sediment is collected on the bottom. Additional depth for the retaining of sediment must therefore be allowed.

If it be desired to plan a basin, capacity 1,000,000 gallons per day (=1.534 cubic feet per second) to remove all particles up to and including .01 millimeter, divide 1.534 by 3×.0005 feet (.0005 foot=0.154 millimeter), giving the cross-section area 1023 square feet. The basin may therefore be made 10 feet deep and 102.3 feet wide. A particle falling .0005 foot per second will fall 10 feet in 5½ hours. Multiplying this by 3 to allow for end disturbances, we have the water remaining in the basin 16½ hours. If flowing .0015 foot per second (3 times the settling velocity of .01 millimeter particles) it will flow 90 feet in 16½ hours, which may be made the length of the basin. Or the basin might have 15 feet effective depth and be 68.2 feet wide and 135 feet long.

If water entered a basin having a cross-section of 1000 square feet through a pipe having an area of 2 square feet, the velocity as it left the pipe would be 500 times the mean velocity in the basin. The kinetic energy corresponding to this difference must be destroyed in some way. Nature's way would be to use it up in friction caused by currents and vortex

motions. These greatly interfere with sedimentation and are to be prevented as far as possible. This is ordinarily accomplished by reducing the entrance velocity and by retarding the currents. To reduce the entrance velocity the water may first enter a chamber extending across the end of the basin, and leave this for the basin over a submerged weir. If the weir be submerged one-twentieth of the depth of the basin, the velocity is reduced from $500V$ to $20V$, V being the mean velocity of flow through the basin. Another way is to approach the basin with a large "bell-mouth" which has a width equal to that of the basin at the junction of the two. To compel the current to distribute itself across the full width of the bell mouth, it would be necessary to supply guides as shown in Fig. 2, page 64.

At best there will probably be a current in the basin caused by the velocity of the entering water. If the entering water is colder than that in the basin, it will fall to the bottom, if warmer, it will remain near the surface. In either case it will tend to move at gradually decreasing velocity until it reaches an obstruction, by which it will be deflected vertically, or horizontally, or both. If no such obstruction be presented, there may be a rapid surface flow from the entrance weir to the effluent weir, the rest of the water remaining practically motionless or swirling in eddies. The designing of obstructions (called baffles) is one of the most important parts of designing a sedimentation basin, and one concerning which there are few known principles that are generally accepted. Any baffle contracts the area of cross-section and therefore theoretically increases the velocity of flow, while the aim is to diminish it. Baffles must be so designed that the beneficial results outbalance this objectionable one. This can be accomplished, and well-designed baffled basins do as much work as much larger ones without baffles. Instead of baffling, some plants have been built with a series of basins through which the water flows in succession.

It seems to have been demonstrated that it is very desirable to prevent the entering water from mixing with that from

which most of the sediment has been removed. This should be one of the aims in baffling.

Since 1.0 millimeter matter settles twelve times as fast as 0.1 millimeter matter, a basin designed with the latter as the limiting size will theoretically retain all of the 1.0 millimeter

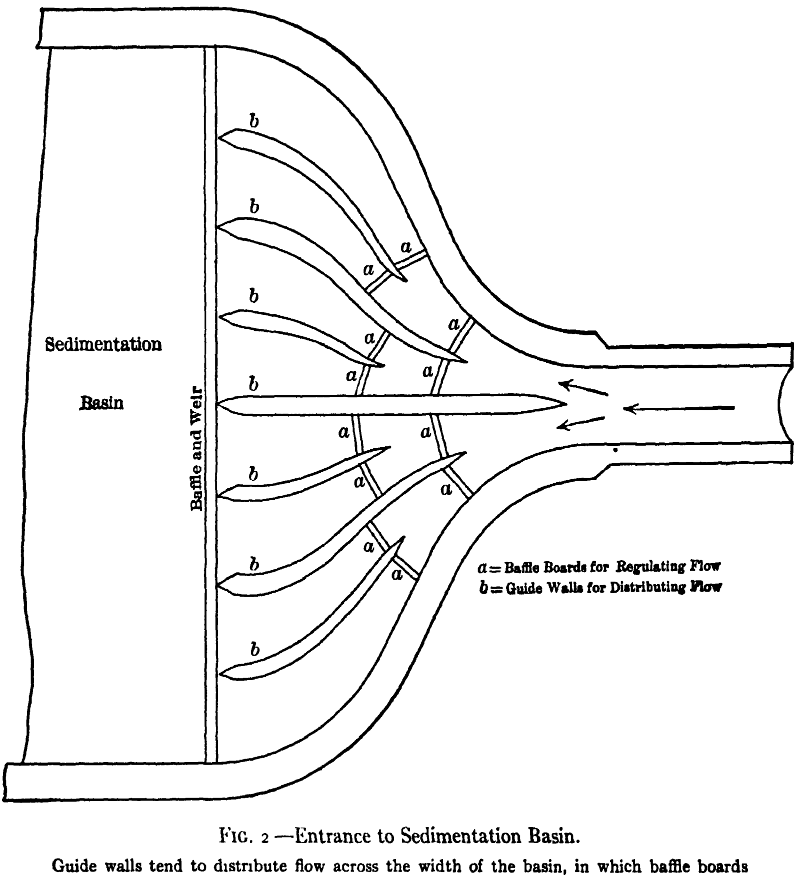


FIG. 2 —Entrance to Sedimentation Basin.

Guide walls tend to distribute flow across the width of the basin, in which baffle boards can assist.

matter and coarser in the first one-twelfth of its length. It is the general experience that the coarse sediment is found near the entrance end. Also in most waters the greater bulk of the suspended matter consists of the coarser particles. If it were all of the same size, the sediment would theoretically

be spread uniformly over the bottom; and this has been the experience in some if not in all plants.

It is evident that each sedimentation basin should be designed expressly for the water to be treated, after making a study of such water during at least one cycle of seasons to learn

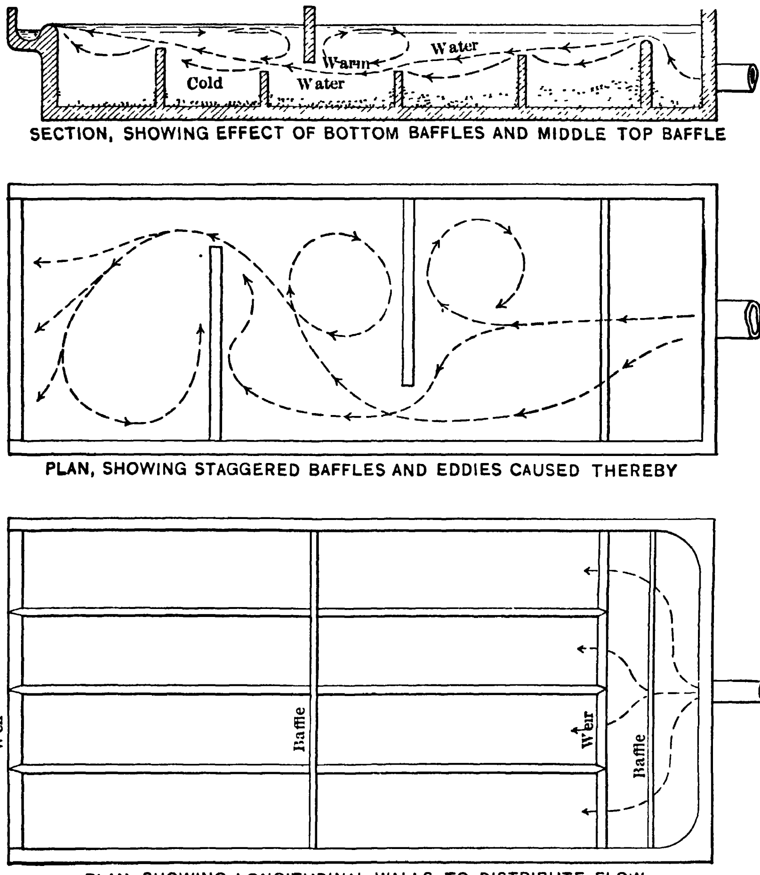


FIG. 3.—Baffles in Sedimentation Basins.

the relative amounts of the different sizes of particles that constitute the suspended matter and the settling velocity of each. Study should also be made of the relative cost of removing the finer matter by building larger basins, by using coagulant, and by straining out such fine matter in filters, respectively.

The sediment that collects in sedimentation basins may be enormous in amount, and require frequent removal. St. Louis in 1915–1916 intercepted in her sedimentation basins 294,000 tons of matter from 29,823 million gallons of water. Of this, 134,200 tons was removed from the basins by teams and the remainder was flushed out into the sewers. The basins are cleaned on the average about once in nine months at a unit cost of 2 cents a cubic yard. This is perhaps the muddiest water in the country that is purified. Ordinarily a deposit of an average depth of 1 or 2 feet is all that need be provided for between cleanings.

Treatment of water for impurities that occur only at long intervals, such as excessive turbidity occasioned by unusual rainfall, may be avoided by providing sufficient storage to tide over several days and taking none of the highly polluted water during that period. This, of course, necessitates ability to treat (and pump, if necessary) at a rate greater than the average, by operating either more units of the plant or more hours a day.

Art. 17. Coagulation

According to Table No. 7, no particle smaller than .003 millimeter will settle to the bottom of an ordinary basin in twenty-four hours. It is seldom desirable to make a sedimentation basin large enough to continue sedimentation longer than this; on the other hand, much suspended clay and most bacteria are smaller than .001 millimeter. (Silt of one-fifth this size is found in the Mississippi river.) If these fine particles can be collected into groups or composite masses which present a smaller area of surface per unit of volume, they will settle more rapidly, and their removal become practicable. This is accomplished by the use of coagulants.

The common coagulants are soluble salts of aluminum, iron, copper, zinc, and some other metals that react with hydroxides, carbonates and bicarbonates of the alkalies and alkaline earths to form an insoluble gelatinous substance which at first permeates the water, then breaks up into flakes about the size

of a pin-head, which form about any particles of silt or bacteria present as nuclei. These flakes have a very fine sponge-like structure, which enables them to absorb coloring matter and gases in solution. These flakes or flocs settle slowly, but attract and attach to themselves other particles of silt and other flocs, until masses of quite considerable size are formed, which settle to the bottom in a few minutes. There generally remain other flocs, however, which increase in size but little and settle slowly.

The metallic salts generally used are those of aluminum and iron (commonly as sulphates) because these are cheaper and because the others are more or less poisonous. Alkaline salts needed to effect the reaction are generally found in the water to be coagulated; but if they are not present, or are in too small quantities, alkalies must be supplied, lime or soda being used for this purpose. Sulphate of aluminum forms a white precipitate, and sulphate of iron a greenish-brown one. Sulphate of iron is not acted upon so quickly by the natural alkalinity of most waters as is aluminum, and when the former is used, lime is generally added to cause a more rapid formation of the iron hydroxide. Iron sulphate costs about half as much as aluminum sulphate, and is heavier and consequently settles more rapidly. But it requires more expert manipulation, and is therefore best adapted to large plants where experienced chemists are on hand to supervise the dosing, and to quite muddy water where the amounts of coagulant required are considerable and the cost of it is consequently an important item.

Aluminum sulphate comes in lumps about half the size of a hen's egg, and may be purchased by the car-load or in barrels of about 370 pounds net. It is generally specified to contain not less than 17 per cent of water-soluble alumina. Alum combined with lime forms soluble calcium sulphate, which causes permanent hardness to the extent of 10.4 parts per million for each grain per gallon of aluminum sulphate used. If soda ash is used to supply the needed alkali, sodium sulphate is formed, which is unobjectionable. Reacting with either soda ash or alkalinity, alum also produces carbonic acid. Not only is carbonic acid corrosive, but algæ are more likely to grow in

water containing it when it is stored in open reservoirs. Soda ash used in sufficient quantity to neutralize the carbonic acid is the most desirable treatment and also the most expensive. If there is not sufficient alkalinity, present in the water or supplied, to react with the aluminum sulphate, basic sulphates will form and, as these are soluble, no coagulant will form.

One grain per gallon of aluminum sulphate will remove about 10 parts per million of color, although this varies with the nature of the coloring matter. For removing turbidity, at least 0.3 grain per gallon should be used, and this should be increased with the turbidity up to a maximum of 2 grains, some of the aluminum sulphate being absorbed by the clay in suspension. The amount required by each water must generally be determined by trial, the completeness of removal of the contained turbidity and bacteria being the test. If no flakes appear in the treated water, either the water is not sufficiently alkaline or more alum should be used. If the flakes are large and feathery, too much alum is being used. Each grain of alum per gallon of water requires 10 parts per million of natural alkalinity, and the water should contain an excess above this of at least 10 parts. If this amount is not present naturally in the water, lime or soda ash must be added to bring the total up to this amount.

In applying alum it is generally dissolved to form a 3 to 6 per cent solution, and this is fed into the water at such rate as to give the desired dose. Or the alum may be powdered and fed into the water dry.

Lime, if bought as quicklime, must be slaked in iron tanks with about four times its weight of water for thirty to sixty minutes, then diluted further and fed into the raw water through a grid or other device for distributing it diffusely. If hydrated lime be bought, this does not require slaking and can be made directly into a solution for dosing, or can be powdered and fed dry.

Soda ash is a fine white powder which dissolves readily, is convenient to handle, prevents after-precipitation of calcium carbonate, and if used in proper amount prevents the formation of free carbonic acid. But it costs four to six times as much as lime.

When sulphate of iron is used as a coagulant, lime is always

used also; and lime, being slow in action, is apt to cause after-precipitation unless the coagulated water is kept for at least four to six hours in the sedimentation basin. If the water contains organic coloring matter, the iron sulphate may give it a black tinge. It is difficult to use it successfully with soft waters. It is therefore best adapted to turbid, alkaline water. There must also be sufficient oxygen in the water to render all the iron insoluble, otherwise it will remain in the clarified water. About 0.4 or 0.5 grain of lime is used with each grain of iron sulphate. There must be sufficient to combine with the iron sulphate, remove any carbonic acid present, and give an excess of 1 to 5 parts per million. An excess of lime must be avoided; also an excess of iron, which may render the water acid. The ferrous hydroxide coagulum is very delicate, and the water containing it must not be violently stirred. The amount of iron sulphate required varies from 1 grain per gallon when the water has a turbidity of 100, to 5 parts for a turbidity of 2800.

Even when coagulant has been added, it may take several days for all the suspended matter to settle out; and the providing of several days' storage capacity will generally involve great expense. In the majority of cases it is less expensive to complete the removal of the suspended matter by straining through a filter. Just how much it is most economical to remove by sedimentation and how much by filtration will vary with the nature of the suspended matter and local conditions that affect construction costs; and each case must be decided separately.

In any event, the water must pass slowly through a basin to give the chemicals time to react, lime especially requiring a half hour or more for this; otherwise coagulation and sedimentation may take place after the water has passed through the filter. When used for securing the reactions, the basin is called a coagulating basin; but there will also occur in it sedimentation of the coarser matters. If there is much matter that will settle rapidly, it will generally be economical to permit this to settle before the coagulant is added, since the amount of coagulant required is governed by the total amount of suspended matter present in the water.

Water should not be pumped while coagulation is taking place, as this interferes with the agglomerating action; but from the time the coagulant is added until after the water has passed through the filters (if the water is filtered), the water should not be agitated, although a gentle motion is believed to aid flocculation.

There is a popular prejudice against putting "chemicals" in water, due largely to ignorance of two things—that normal water ordinarily contains matters as truly chemicals as those used in treating water—salt, iron, and lime, for instance; and that the amounts used are so minute that a consumer would need to drink hundreds of gallons of water to obtain a medicinal dose. In fact, no free alum passes through a filter if it is properly operated, and the same is true of iron in the iron sulphate, while the sulphuric acid formed is neutralized by the lime or natural alkalinity of the water. Perhaps the most objectionable constituent resulting from chemical treatment is carbonic acid, which is imbibed fearlessly in soda water and bottled carbonated waters. In fact, the last-named contain hundreds of times as much chemical adulteration as probably ever flows from a water-purification plant. As to matters carried in solution by normal waters, a typical unpolluted hard water was found by analysis to contain 6 parts of sodium sulphate per million, 57 of calcium sulphate, 8 of sodium and potassium chloride, 7 of sodium and potassium nitrate, 1 of bicarbonate of iron, 170 of calcium bicarbonate, 135 of magnesium bicarbonate, and 17 of silica—several hundred times as much "chemical" matter as the effluent of any well-operated coagulation plant.

Art. 18. Filtration

Rapid Sand Filters. When water has been treated with a coagulant and this has collected to itself practically all of the suspended matter in the water, all of this matter that has not previously settled out may be removed by passing the water through a sand bed, which strains out the flocks of coagulated matter. Most of these are caught in the top layer of sand, although some of the finer matters penetrate to a depth of

several inches. In addition, after a layer of the coagulant jelly has collected on the filter, the remaining water passes through the fine pores in it, and in doing so practically all of the finest suspended matter which may not have been enmeshed by the coagulant in the basin is caught and held by this surface jelly.

The sand used has an effective size * usually between 0.4 and 0.6 millimeter. This is placed to a depth of about 26 to 30 inches, and rests upon layers of gravel graded in size from $\frac{1}{8}$ inch or less at the top to $\frac{1}{4}$ inch or more at the bottom, and having a total depth of about a foot. The gravel rests upon a bottom in which is some system of strainers which will admit the water uniformly at all points and keep out the gravel. The water that is being filtered is forced through the sand by keeping its surface about 2 feet above the top of the sand.

As the water passes through this at the rate of 100 to 150 million gallons per acre per day, or about 12 to 19 vertical feet per hour, silt and coagulant collect at the top in increasing amount, until finally water passes through it too slowly for economical working, when the sand is washed by reversing the flow of water, the upward flow carrying out most of the silt and coagulant jelly, the dirty water being drawn off to a sewer or other outlet. It is not desirable that the sand be washed perfectly clean, but thin films of the jelly left on the sand grains help to clarify more perfectly the first of the next run of water, before it has deposited its own jelly layer. (St. Louis applies an aluminum sulphate solution to the water on top of a filter after cleaning and before renewing filtration, to form a coagulant coating rapidly on top of the sand.)

Washing takes ten to twenty minutes; then the upward flow is stopped, the sand is allowed to settle for a minute or two, and filtering is begun again. This is the general action

* The effective size is defined as "such that 10 per cent of the material is of smaller grains and 90 per cent is of larger grains than the size given. The results obtained at Lawrence indicated that the finer 10 per cent have as much influence upon the action of the material in filtration as the coarser 90 per cent." The uniformity coefficient is "a term used to designate the ratio of the size of grain which has 60 per cent of the sample finer than itself to the size which has 10 per cent finer than itself."

Recently an effective size of 0.35 to 0.45 has come into common acceptance for filters.

of the rapid sand filter. Air also is blown through some filters in washing them, to assist in stirring up the sand.

Nearly all rapid filters are built in a number of units, rectangular in shape and uniform in size, the dimensions varying from 15 to 50 feet in the different plants; concrete being used for the building material. The standard plan is to arrange the filter beds on two sides of a central gallery, in which gallery are pipes (or conduits, if the quantity treated is very large) for bringing the coagulated water to the beds, removing the filtered water, supplying filtered water and air for washing, and drains for removing the dirty wash-water. Each of these pipes is connected to the proper part of each bed by a branch which is controlled by a gate valve. The coagulated water generally flows from the feed main into a gutter that extends the full length of the bed, over whose sides as weirs it flows onto the sand with a minimum of disturbance. The filtered water passes by gravity through the strainers at the bottom of the sand bed into the filtered-water main and in this flows to a clear-water basin. (In some plants the clear-water basin is under the filters, partly for economy of land and construction, partly because here it is kept cool and light is excluded to prevent growth of algæ.) In washing, filtered water is forced through a wash-water main into and through the strainers from below and up through the sand, the volume used being sufficient to bring the sand to a semi-suspension and wash out the dirt, but not sufficient to carry the sand out into the drain. In many plants the bonding of the sand together by dirt and jelly is broken up by blowing air through the sand before the wash-water is turned on, which permits lower velocity of wash-water—an advantage because of the tendency of high velocity to raise the gravel and allow the sand to fall through it onto the strainers. Where air is not used, it is customary to place a brass screen between the gravel and sand to prevent the gravel rising (which screen may corrode and break, as happened at Cincinnati).* The older filters (and some of the smaller

* Plans for Baltimore filters made in 1922–23 omitted both brass screen and strainers.

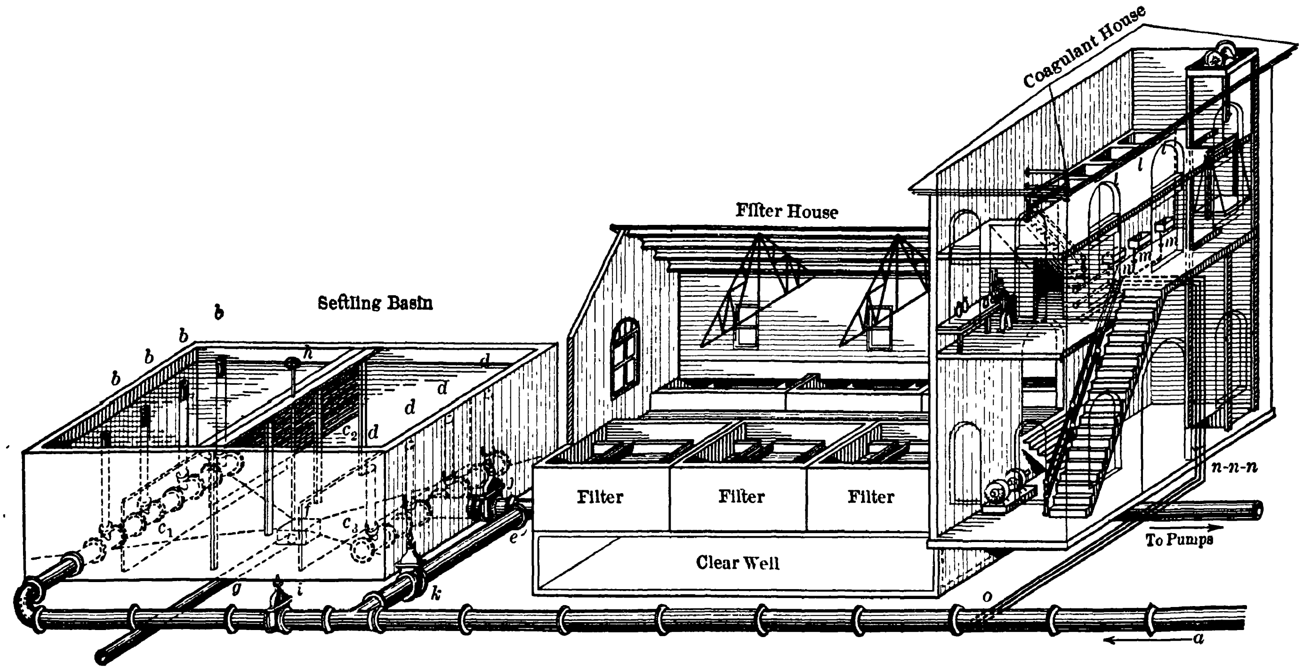


FIG. 4.—General View of a Rapid Sand Filter Plant.

From "Water Purification Plants and Their Operation," by Milton F. Stein; by permission of the author.
a, raw water main, *b*, risers from inlet manifold, *c*, baffles; *d*, risers from outlet manifold, *e*, settled water main; *f*, sump, *g*, drain, *i*, *j*, *k*, valves; *l*, solution tanks, *m*, orifice boxes, *n*, solution pipes.

new ones) were circular in plan, and revolving rods or raking bars were used to stir up the sand in washing.

Filtered water should be used for washing, otherwise bacteria and other impurities might be carried into the bed by the wash-water, and be washed out into the clear-water main

Fig. 5.—Strainers in Floor of Rapid Filters, Baltimore, Md.

Filtered water leaves through openings in the strainer plates that cover the long narrow slots and communicate with the wide central drain. Wash water enters through openings in this drain, a deflector plate over the opening distributing the water so that it will pass under and rise through all the strainer plates. A deflector plate is seen at the far end of the central drain. The troughs spanning the filter lead away the wash water, which flows over their edges as weirs, and the untreated water enters by the same troughs or "gutters."

when filtering is resumed. The wash-water may be supplied by special pumps, may be drawn from the pumping main feeding the city distribution system, or from a tank elevated to the height necessary to give the desired pressure, this tank being filled by pumps or from the pumping main. The last is the best, at least for large plants.

There are numerous details for facilitating operation, many of them patented. These include strainer systems, which

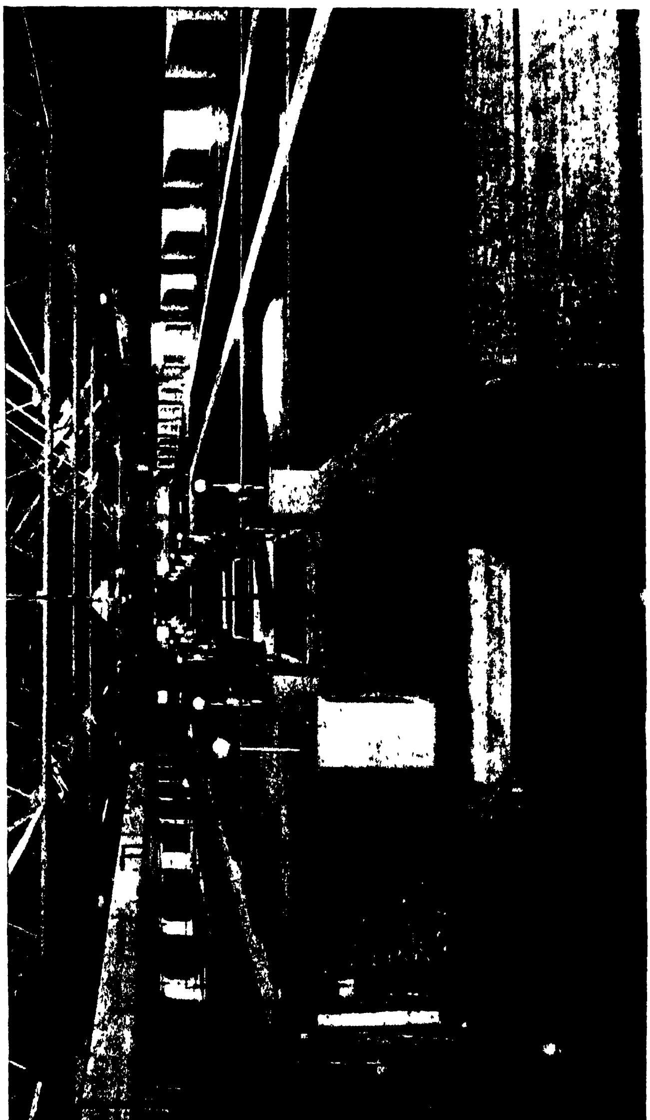


FIG. 6.—Rapid Filters at Louisville, Ky.

This shows the operating gallery, with glimpses of the pipe gallery beneath. The entire area of all the filters is covered by the filter house and is thus visible; but in most plants the building covers only about one-third of each filter, the other two-thirds extending beyond the building and being roofed over, as in Fig 4. On the operating gallery floor are seen the operating stands opposite their respective filters.

furnish outlet for the purified water and inlet for the wash-water and air; rate-of-flow controllers for maintaining the

flow through the filter constant at any desired rate; a draft tube for creating negative pressure ("suction") in the outlet of the filter; valves operated hydraulically, etc. The larger plants have for each filter an operating table at which are six levers, each of which controls a valve (each filter has an influent, effluent, wash-water, sewer, air, and filter-drain valve); a loss-of-head gage showing the friction head for the filter; and possibly rate-of-flow indicators, and gages showing pressure in wash-water and air pipes. There should also be provision for obtaining a sample of the effluent from each filter at any time.

The result obtainable by rapid sand filters may be illustrated by the St. Louis filters, which are among the most complete and largest in the country. During the year 1920–1921 the following results were obtained:

TABLE NO. 8A

RESULTS OBTAINED BY ST. LOUIS PURIFICATION PLANT, 1920–1921

NOTE: The water is treated with lime and ferrous sulphate and passed through sedimentation basins. Aluminum sulphate is then added and it is passed through rapid sand filters. Chlorine is applied to the effluent. The filters started in regular operation in the middle of May, 1915. The fiscal year ends March 31

Of the letters used for column heads, *R* means raw river water; *A*, water after settling and as applied to the filters, *F*, effluent from filters; *M*, water as it enters the mains. Figures are the averages for the several months.

Month.	Suspended Solids.		Turbidity.				Color.		Total Hardness.	
	R	*M*	*R*	*A*	*F*	*M*	*R*	*M*	*R*	*M*
April	1440	0	1500	7	0	0	28	10	145	96
May	2390	0	2200	8	0	0	27	10	180	105
June	3150	0	2500	14	0	0	27	11	190	105
July	3240	0	3000	27	0	0	34	16	165	95
August	960	0	1020	24	0	0	30	13	175	90
September	1210	0	1300	21	0	0	26	11	160	80
October	350	0	370	14	0	0	25	10	190	105
November	255	0	270	8	0	0	24	9	205	115
December	370	0	340	8	0	0	22	9	225	120
January	240	0	200	10	0	0	20	10	240	140
February	740	0	620	9	0	0	20	10	210	120
March	1070	0	1010	10	0	0	22	10	183	102
Average	1255	0	1200	13	0	0	25	11	180	106

Sulphates increased from 73.8 parts per million in the raw water to 81.1 in the filter effluent because of the use of sulphates of aluminum and iron as coagulants. Chlorine averaged 10 p. p. m. in each. Albuminoid ammonia decreased from 0.900 to 0.107. Dissolved solids decreased from 305 to 205.

Table No. 8B

BACTERIAL RESULTS OF ST. LOUIS PURIFICATION PLANT, 1920–1921

Month.	Numbers of Bacteria per Cubic Centimeter									Number of B. Coli per Cubic Centimeter.				
	On Gelatine at 20° C.				On Agar at 37° C.									
	River.	Applied.	Filtered	To Mains.	River.	Applied.	Filtered.	To Mains.	To Consumers.	River.	Applied.	Filtered.	To Mains.	To Consumers.
April..........	20,600	2300	310	80	6,000	55	13	10	8	5 9	0 0508	0 0037	0.0007	0.0000
May..........	27,000	3450	400	110	9,600	150	26	17	10	15 4	.2193	.0353	.0298	0293
June..........	15,200	365	75	34	12,000	125	41	70	28	39 6	.7862	.0704	.0181	.0800
July..........	9,200	80	20	15	12,500	170	55	55	48	41.0	.5320	.1221	.0253	.0323
August........	5,300	45	15	48	11,000	80	25	60	49	27.0	.0691	.0135	.0251	0175
September.....	8,300	50	13	24	14,300	100	22	28	29	34.8	.3141	0338	.0523	0285
October.......	4,500	85	16	30	7,100	130	24	95	27	10 4	.5743	.1020	.0925	0330
November.....	5,500	120	27	15	5,000	75	14	11	10	7 3	.5544	.1084	0073	0292
December.....	6,350	250	60	13	3,550	75	12	7	6	10 4	.7762	.1467	.0093	0019
January.......	12,300	470	85	17	2,100	60	14	7	5	3 4	.1841	.1003	.0020	0000
February......	15,000	360	50	10	4,300	70	13	10	6	3 6	.1242	0307	.0014	0000
March.......	16,100	410	54	65	6,600	70	15	15	10	4 8	.1289	.0514	.0016	0014
Average......	12,100	670	95	41	7,850	95	23	32	20	17 0	3613	0685	0222	0209

In the New Orleans filters, which have been operated since 1910, the only sedimentation is in the grit chamber and coagulating reservoir. The average figures for 1915 for raw water, effluent from coagulating reservoir, and filter effluent, respectively, were: Turbidities—900, 25, 0. Alkalinities—91, 42, 35. Bacteria—1900, 170, 15. The maximum number of bacteria in filter effluent was 250. There were used 4.75 grains of lime and 0.60 grain of iron per gallon. The average length of run between cleanings was 156 hours; 0.4 per cent of the water filtered was used in washing the filters.

The rapid filters above described are known as the gravity type. A number of municipal and more commercial plants use pressure filters. These act on the same principle as gravity filters, but the sand bed and strainers, being under pressure, are enclosed in a steel cylinder, for which reason the capacity of each unit is generally limited to half a million gallons a day. (The largest plant to date—1916—is that at Atlanta, Ga.—21 million gallons; but of the 151 in service, 140 are 5 million or less capacity.) This cylinder is generally interpolated in the pumping main and the water in it may therefore be under 100 pounds pressure or more; but the pressure forcing it through the sand (the difference of pressures at inlet and outlet) should be no greater than in a gravity filter. The cylinders are sometimes vertical, but more often have the axis horizontal.

The advantages of pressure filters are that they cost considerably less than gravity; can be installed more quickly—the smaller sizes are kept in stock; are more accessible for repairs; are more economical of floor space—may even be placed one above the other. They often are used without either coagulation or sedimentation basins, and thus the low lift from river to basin is eliminated; but with many waters the sedimentation is desirable, and the omission of it detracts from the efficiency of the filter. Even when such basins are supplied, the pumping of the coagulated water before it has been filtered is said to break up the flock. Another disadvantage is that devices for controlling the rate of filtration and the rate of supplying coagulant have not been perfected. Because of

this, of the high pressure in the tank, and difficulty of watching the operation, there is more liability of overworking the filter, of flushing out sand when washing it, and in other ways of producing inferior results if the filter is not carefully and intelligently watched, than in the case of gravity filters. Many engineers believe that, with improvement in the mechanical features, the pressure filters can be depended upon for effective service, especially since any lapse in bacterial removal can be remedied by applying a disinfectant to the effluent; also that there are certain conditions where a pressure plant could be located much more easily than a gravity plant.

Slow sand filters were in use for many years before rapid sand filters were devised. They have been used in England since 1829 and in this country since 1872; while rapid sand filters were first used for municipal supplies in 1885, and were first established on a sound scientific basis by the investigations of Geo. W. Fuller at Louisville in 1895–1898, and by others during the next few years. A slow sand filter consists of a series of water-tight basins, each usually between $\frac{1}{4}$ acre and 1 acre in area, filled with sand to a depth of $2\frac{1}{2}$ to 5 feet. This sand rests upon graded gravel, as in the rapid filters, and the gravel upon a floor sloping to drains placed at intervals to remove the filtered water. In general the gravel is placed in layers 2 to 8 inches thick, the grains or stones in each layer having about one-third the diameter of those of the layer beneath. In the Albany filters the bottom layer was 7 inches thick, of stone 1 to 2 inches in diameter; the next of $\frac{3}{8}$-inch to 1-inch stone; the next varying from $\frac{3}{8}$ inch to the size of the sand used (say $\frac{1}{64}$ of an inch); each of the upper layers being $2\frac{1}{2}$ inches thick.

The underdrains may be of sewer pipe, with joints laid open and wrapped with cheese-cloth; or split pipe laid, concave surface down, on a smooth cement floor (frequently bedded in a thin layer of fresh mortar), with joints open; or special box drains with slotted top surfaces. Whatever the detail, they should be of abundant capacity for removing the filtered water at a rate at least double that for which the filter is designed.

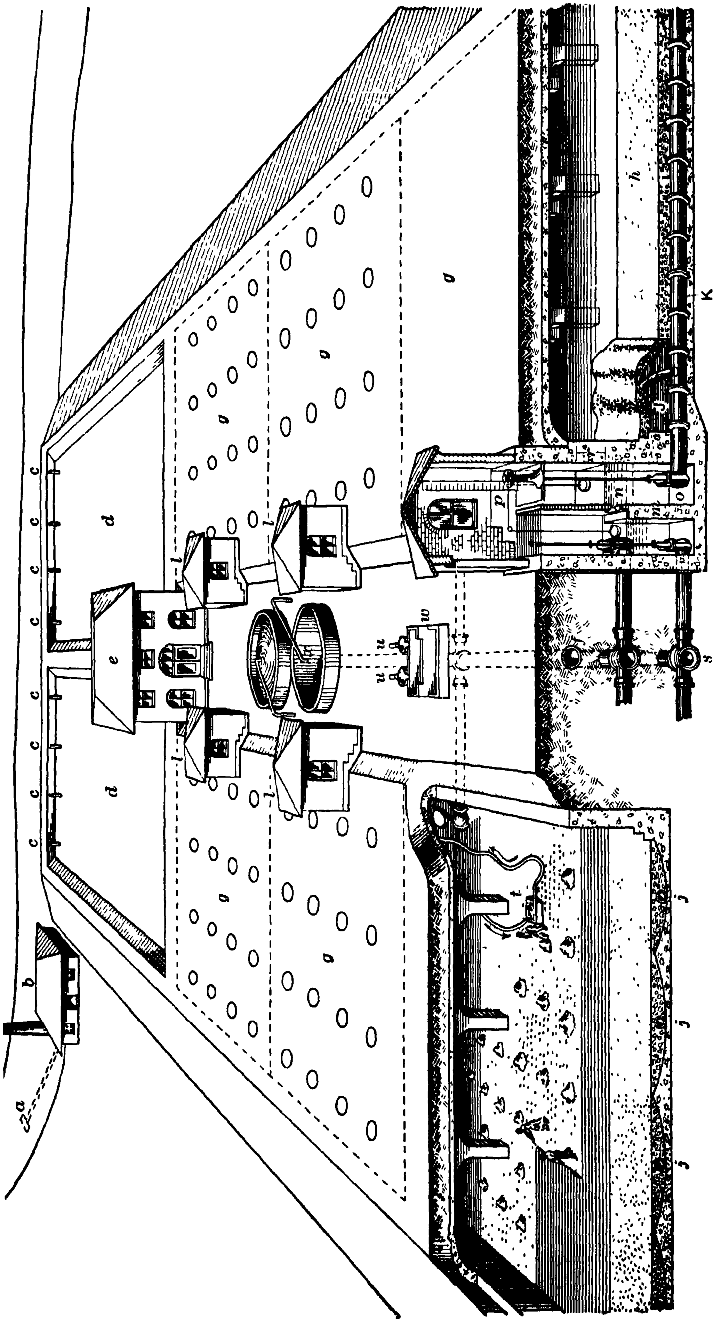


FIG. 7.—General View of Slow Sand Filter Plant.

From "Water Purification Plants and Their Operation," by Milton F. Stein; by permission of the author.

The side-walls of the filter basin, and any piers therein for supporting a roof, generally have offsets in their surfaces (probably a rough surface also would be advantageous) to prevent water that passes through the filter from following the wall and thus reaching the underdrains without passing through the sand. The walls and bottom should be water-tight, to prevent loss of water and even more to prevent infiltration of impure ground water.

Most filters are covered over, especially those in climates where solid ice forms, to prevent the beds from freezing or the water above them. Roofs also prevent the growth of algæ, which have been known to clog filters badly. They add about $15,000 to $20,000 an acre to the cost of filters, but will generally save many times this in operating expenses. The almost invariable practice in this country is to cover the filter with a groined * roof of concrete, resting on concrete piers, which in turn rest on a concrete bottom of inverted groined arches. There are different methods of cleaning slow sand filters, and there must be provided in the construction the runways, pipes, etc., necessary for performing the cleaning process adopted.

A constant depth of water over the sand is maintained by a float-valve that controls the inlet pipe. The rate of filtration is kept constant by varying the head (between inlet and outlet of filter) as the resistance of the sand bed varies, and this is effected at the outlet, several contrivances being available for this purpose, some automatic and others regulated by hand. Some of the automatic ones are based on the general principle of maintaining a constant head on a weir over which the effluent flows in leaving the filter; others use a venturi meter on the effluent line to operate a valve controlling the effluent, etc.

In slow sand filters the water is passed through the sand at a rate between $2\frac{3}{4}$ and 6 million gallons per acre per day, or about one-fortieth to one-twentieth the rate of rapid sand filters. The sand generally has an effective size of 0.3 to 0.4 millimeter, or somewhat finer than is used in the rapid filters. The water is not coagulated first, but should be passed through a sedimentation basin if turbid. As the water moves slowly

* Since 1918, three or more plants have been built with reinforced concrete flat-slab roofs.

through the filter (the actual velocity through the interstices averages about a foot in ten to twenty minutes) it deposits any suspended matter that it may carry on the tops of the sand grains. Also there accumulates around each sand grain a jelly-like film of bacterial origin. Most of this gelatinous growth and of the deposited sediment collect in the top half-inch or so of the bed, and combined form what the Germans call schmutzdecke, which term has been adopted in this country. This schmutzdecke acts as a finer straining medium for removing bacteria and other suspended matters; and it is probable that it attracts and holds such matters by surface adhesion and possibly in other unknown ways. Not until this is formed does the filter reach its full efficiency; and this requires several days and sometimes weeks. During the first two or three days 75 per cent of the bacteria in the applied water may appear in the effluent, and it may be two or three weeks before 99 per cent or more are removed by the filter. (The diameter of the interstices between clean sand grains is several hundred times the thickness of a bacterium.)

After passing through the schmutzdecke, in which it has left 75 to 85 per cent of the bacteria and an even larger percentage of the suspended mineral matter, the water passes slowly through the interstices of the remainder of the sand-bed, which is generally between 2 and 4 feet deep. Here also is bacterial jelly, but in less amount, and here takes place bacterial oxidation of the organic matter, both that in suspension and that in solution, into the form of nitrites and nitrates. This oxidation requires time, and this is a chief reason for fixing the rate given as the maximum desirable. It also explains why such oxidation does not take place to any great extent in rapid sand filters, another reason for the latter being that it requires several days for the nitrifying bacteria to develop and become established in a filter, and the rapid sand filters are generally washed daily or oftener. Another reason for fixing the low rate is that if by force a high velocity is obtained, the schmutzdecke is apt to be broken and unpurified water pass through the filter. In addition to the effects and

causes referred to, it is probable that the destruction of the bacteria is assisted by the removal of their nitrogenous food-matter by oxidation. If the raw water contains little oxygen, it may be necessary to let the bed drain out at intervals of a few hours and fill with air, when the water is again turned onto it from the top.

The schmutzdecke and bacterial jelly naturally suggest the coagulant of the rapid sand filter, and it is true that both perform the common function of straining. But one is chemical in origin, the other is biological. The coagulant has already agglomerated a large part of the suspended matter before it reaches the filter; the schmutzdecke forms on the filter and strains out or seizes by adhesion the suspended matter as the water flows through it. Also a certain amount of sediment collects by deposit in the upper interstices of the slow filter, and oxidation of organic matter takes place throughout it, neither of which is possible to any considerable extent in the rapid filter.

As suspended matter collects on and in the gelatinous covering, the pores of this and of the sand become so choked as to interfere with the passage of water through the filter. In fact, soon after the filter is put into service it is found that the head of water on the filter must be increased to produce the required velocity of percolation. The limit of head of water to be permitted with safety is set at from 24 to 52 inches by different authorities. When the limit has been reached the filter must be cleaned, and then "ripened" by securing a new accumulation of schmutzdecke and bacterial jelly before it can be put into service, the water passing through very slowly meantime. The interval between cleanings varies from a week to two months (sometimes even longer), depending upon the rate of filtration and amount of turbidity, algæ or other suspended matter in the water—in other words, upon the amount of suspended matter conveyed to the filter. Almost any practicable head on the filter may be used, in the opinion of some experts, provided the velocity is not too great. Some also claim that the limit of velocity might be raised to even 8 or 10 million gallons

per day, but that to do so would carry the sediment deeper into the filter and thus require the removal of more sand in cleaning, and would require three times as frequent cleaning, so that little, if anything, would be gained, except when the water is quite free from suspended matter and the accumulation of it upon the sand is therefore slow.

Cleaning these large filters by forcing water upward through them seems to be impracticable, and in addition would remove the gelatinous coating on the sand grains and the nitrifying bacteria. The most common method of cleaning is to let the water drain down to a level a few inches below the top of the sand bed, and then remove, by use of broad flat spades, the clogged sand on top of the filter, which is generally ½ inch to 1 inch deep, there being a fairly sharp line of demarcation between this and the sand which is not seriously clogged.

When this sand has been removed, the filter is filled again for operation. In some cases the filter is filled from the top with unfiltered water, and the effluent allowed to run to waste for a few days until a good schmutzdecke is formed. But the more modern plan is to fill the filter from below with filtered water and start it in action very slowly, when (in the opinion of some, at least) it is not necessary to waste any of the effluent.

This removing of the top sand is repeated from time to time until the thickness of the sand bed reaches the minimum that is considered safe, when the bed is refilled to its original depth with clean sand. The minimum safe depth is probably between 24 and 18 inches; tests to determine this point should be made for each plant, stopping at a point just above that where the purification effected (as shown by analyses of the effluent) begins to be less than that secured from the deeper beds.

The sand removed in cleaning was, in the earlier plants, taken by wheelbarrows up a runway to a pile outside, where it was cleaned so as to be used again. (In most localities the cost of sand such as is necessary for filters is so great that it is cheaper to clean the old at considerable expense than to purchase new.) Later plants remove the sand through pipes.

by water carriage. For this purpose there are provided in each filter a water-pipe and a sand-pipe, each about 4 inches diameter, with outlets at intervals for hose connections. A portable sand ejector is moved from place to place on the filter and at each position is connected with these two pipes by lengths of hose. This ejector is somewhat similar to a steam ejector, water flowing through under pressure and sucking into it the dirty sand that is shoveled into water in the box that contains the ejector. The sand is carried through the sand-pipe

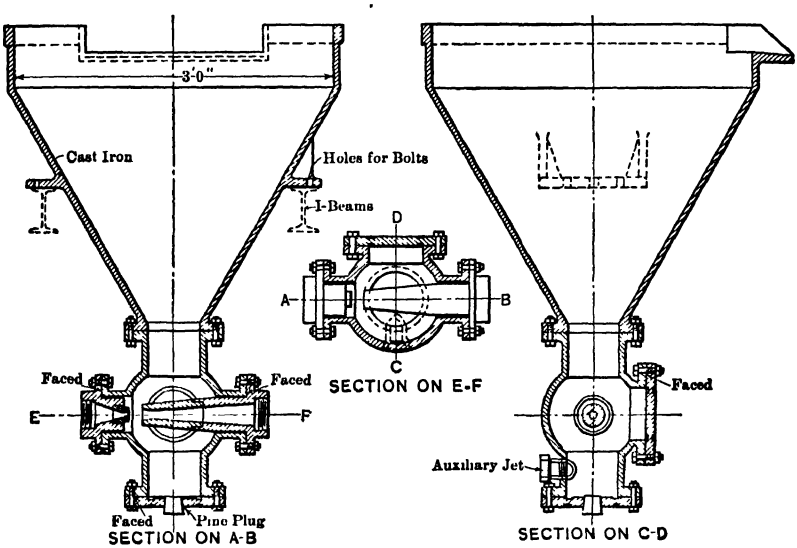


FIG. 8.—Sand-washing Hopper.

in semi-suspension in the water to a sand-pile or bin outside, or more commonly to a sand washer.

There are several designs of sand washer, the general features common to all being a box into which water enters from the bottom and overflows at the top, while the dirty sand is discharged into the box from above from the sand-pipe just described, or by shovel from a bin. As the sand settles to the bottom, the rising water washes it and carries away the dirt as it escapes at the top of the box. The sand that falls to the bottom is either removed through an outlet or is seized by

another ejector and carried to sand bins, or to another washer where it is again washed in the same way.

In replacing sand in the beds, it may be wheeled in and spread by shovel, but the better way is to fill the filter with filtered water and discharge the sand into this through pipes and hose (through which it is carried by water), the end of the hose being moved about on a raft so as to distribute the sand evenly. The water is then drained down to expose the surface, and this is leveled off.

The water carriage method of handling sand, and sand washing, are impracticable when the temperature is much below freeezing. In such weather the sand that is scraped up may be left in piles on the filter (which, however, reduces the filtering area), or it may be wheeled out in wheelbarrows. Washing is generally omitted in such weather, the accumulated sand being washed at the first warm spell.

Another method of remedying clogging is to wash the sand in the filter by the "Brooklyn" method (first used in that city). In this, the water is drawn down until 1 inch deep above the sand. Unfiltered water is then run over the surface of the bed and escapes through an outlet drain, the sand meantime being agitated with long-toothed garden rakes. The water flows over the bed at a rate sufficient to carry away the dirt but not the sand. The bed is washed in strips 10 or 12 feet wide, to which strips the water is confined by boards on edge driven into the sand in two continuous rows and thus forming the sides of a flume.

An appliance known as a Nichols separator has been used for washing the sand in the bed and restoring it at once to the same bed. This is essentially a sand-washer that is set on the filter, washes the sand by water piped in, and discharges the dirty wash-water outside the filter through other pipes. Another machine known as the Blaisdell sand-washing machine has been used in three or four plants. A revolving head carrying hollow vertical rods or teeth is carried on a traveler that moves on rails on the side walls of the filter; the traveler being driven and the head revolved by electric motors. The teeth

are lowered into the sand and water forced through them under a pressure of 10 to 20 pounds, the head carrying the teeth meantime revolving and the whole machine traveling forward at a speed of about 10 feet per minute. This stirs up and washes the sand without removing it from the filter.

Instead of removing a sand layer whenever the filter becomes clogged, several plant superintendents rake the sand at intervals, loosening up a thin layer by use of garden rakes with teeth 1 inch long. A first raking is almost as effective as scraping; a second is less so; and when clogging occurs after the second raking, it is more economical to scrape the filter. In this scraping as much sand is removed as in three scrapings without raking; but the cost of two rakings and a deep scraping is less than that of three ordinary scrapings.

The time that each filter is out of service for cleaning and that lost in working it up to its normal rate of filtration reduce its average effective working rate for the year by 10 to 20 per cent, and it is considered desirable to provide 15 to 25 per cent greater area than that theoretically necessary for filtering a given amount of water, providing that at least 6 beds be provided. The less the number of beds, the greater must be the surplus area. If there are but two beds, each must have capacity for the total amount filtered.

The results obtained by slow sand filters are illustrated by the Table on page 88.

ART. 19. OTHER PURIFICATION METHODS

Aeration is employed for adding oxygen to water or for releasing gases. It is effected by passing water in thin sheets over weirs or inclined planes, by spraying in the air from fountains, by allowing to flow through perforated trays, to trickle through coke or other coarse-grained filters, to flow down a channel filled with stones which violently agitate it, etc. Probably the fountain is the most effective, but it also uses up the most head. Removing gases requires more thorough agitation than introduction of oxygen. As described above,

intermittent operation of a filter oxidizes the water. There is some advantage in passing air through the water in small bubbles, but this is perhaps the least effective method. The gases most commonly removed by aeration are carbon dioxide and certain hydrogen gases that cause odors in water.

Table No. 9

RESULTS SECURED BY THE SLOW SAND FILTERS AT LAWRENCE, MASS., 1915

Parts per 100,000

New filter operated at 3,000,000 gallons per day, old filter at 1,000,000 gallons.

Sample Taken from	Temperature Fahr. deg.	Appearance.		Ammonia.			Chlorine.	Nitrogen as	
		Turbidity.	Color	Free.	Albuminoid. Total	Albuminoid. Solution.		Nitrates	Nitrites.
Merrimac river	51	0 3	0 35	0190	.0214	0177	.47	021	0002
Effluent from new filter	52	0 0	0 26	0057	0074		43	027	0000
Effluent from old filter	53	0 1	0 31	0150	0081		48	033	0002

Sample Taken from	Oxygen Consumed.	Iron.	Alkalinity.	Soap Hardness.	Bacteria per c.c.			Per Cent of Bacteria Removed.			Percentage of Samples Containing B. Coli.	
					20° C	40° C. Total.	40° C. Red.	20° C.	40 C. Total.	40 C. Red.	1 c.c	100 c.c.
Merrimac river.	64	0540	1 6	1.8	12,000	815	295		. .	. .	100	100
Effluent from new filter.	44	0230	1 4	1 7	50	11	3	99 6	98 1	99 0	15 6	85 8
Effluent from old filter.	45	1070	1 8	2 0	25	8	1	99 8	99 1	99 6	7.5	77 9

Note. The old filter has an earth bottom through which some ground water enters the effluent. It is uncovered. The new filter is covered and has a concrete bottom and side walls.

When iron is present in water as a carbonate or in other easily oxidizable form, it can be removed by applying any of the methods of aeration, which renders the iron insoluble, when it may be removed by filtration. In some plants filtration through a pressure filter without coagulation is satisfactory

for this purpose. In certain cases organic matter in the water tends to hold the iron in suspension, even when oxidized; in which case it may be possible to remove it by use of aluminum sulphate, sedimentation and filtration. Iron present as ferrous sulphate has been removed by the use of lime, aeration, and filtration. Iron and manganese found difficult to remove by other methods have been removed by oxidizing in a coke filter used as a contact bed (filled, allowed to stand full, emptied, and allowed to stand empty, in rotation), then allowing it to settle for an hour in a basin, and filtering.

Softening. Although hard waters are objectionable, comparatively few cities soften their municipal supplies, but the number will probably increase within the next few years. McKeesport, Pa., Columbus and Oberlin, O., Freeport, Ill., New Orleans, La., and St. Louis, Mo., are among the cities that now soften their supplies. Softening is effected by adding to the water calcium hydroxide and sodium carbonate, which cause reactions that change the calcium in solution to insoluble calcium carbonate, and magnesium (also any aluminum and iron present) to insoluble hydroxides. The lime added must be sufficient to provide hydroxyl to combine with the iron, aluminum, magnesium, bicarbonate, and hydrogen radicles, and carbon dioxide. Moreover, if the carbonate radicle in the water plus that formed by the change of the bicarbonate and carbon dioxide is not sufficient to precipitate the calcium present in the water and added as lime, a larger quantity must be provided by the addition of soda ash; this determining the amount of soda ash to be used.

The lime is applied either as milk of lime or, preferably, as lime-water, and the soda ash as a 20 per cent solution dissolved in hot water. After these have been added and thoroughly mixed with the water, the latter is held for at least two or three hours in a sedimentation chamber, both for sedimentation and to insure the completion of the chemical reactions. The water is then filtered to remove the balance of the suspended matters. If it is to be filtered for general purification, the same filters serve both purposes. The presence of precipitate already formed

hastens further formation of precipitate; consequently it is common to apply practically all the chemicals to a part of the water (more than half) and after precipitants have formed in this, to mix the balance of the water with it. The higher the temperature of the water is maintained in winter the more rapid and complete the reaction.

Removal of algæ and their odors from water is often accomplished only with much difficulty. One of the most difficult cases on record is that of Springfield, Mass., where, after several years of experiments, part of the time by engineers of the state board of health, complete success was obtained by aerating, passing intermittently through four filters in rotation, again aerating and filtering.

Color can sometimes be removed by aeration and filtration, but generally aluminum sulphate gives better results. Sunlight will often bleach out a colored water exposed in a reservoir, but this is a slow process. Water from the bottom of deep reservoirs or lakes, where the organic matter responsible for the color has undergone putrefaction, can generally be decolored by filtration without the use of alum; and the sand process will ordinarily remove 20 to 30 per cent of the color in river water and reservoir water that has not putrefied. It is believed that the easy removal of color from deep waters is due to the fact that the organic matter in such waters uses up the free oxygen present, then absorbs more from iron in the ferric state in the bottom sediment, thus changing it to the soluble ferrous state. The iron thus taken into solution is precipitated by absorbing oxygen in the filter, and acts as a coagulant to remove the color.

Stream-line filters. A new type of filter known as the stream-line filter is being developed (1924). It consists of a pack of sheets of paper that is impervious to water and oil and somewhat roughened. Two sets of holes are punched through the pack, the water being introduced under pressure into one set, passing between the sheets to the other set of holes and emerging through them. It is claimed that it will remove suspended matter, bacteria and even color from water.

Art. 20. Disinfection

Disinfection has taken a very important place in water treatment since about 1910. It consists in subjecting the water to some treatment that will kill a large part of the bacteria. It produces no clarifying effect or removal of suspended matter. The only methods that are in general use at present (1924) are based on the action of chlorine in liberating nascent oxygen, a powerful germicide. The chlorine is applied either as hypochlorite of lime or as chlorine gas. (Hypochlorite of soda might be used, but is not in any plants in this country, to the author's knowledge.) Experimental plants have demonstrated the effectiveness of ozone and of violet rays as germicides under laboratory conditions; but no service plants have yet proved successful with respect to both effectiveness and economy. Boiling and distillation will each sterilize water, but are impracticable for the large amounts involved in a municipal supply.

Hypochlorite of lime (also known as "bleach" or chloride of lime) is about 35 per cent calcium chloride, 35 per cent calcium hypochlorite, and 30 per cent impurities. The first and last are inert when water is added, but the calcium hypochlorite, acted on by free and half-bound carbon dioxide in the water, forms hypochlorous acid. This acid is unstable and readily gives up, to any organic matter present, oxygen in the atomic or nascent state. The germicidal effect is produced if no carbon dioxide is present, but more slowly. These reactions require time, thirty minutes being a minimum, the time increasing as the temperature and amount of free carbonic acid decrease. The oxidizing strength of hypochlorite is expressed in terms of "available chlorine," which is ordinarily 37 to 40 per cent of the commercial bleach.

The amount of available chlorine used depends upon the amount of impurities in the water and the approximation to complete sterilization desired, limited by the danger of giving an unpleasant taste to the water. The only safe rule is to test the germicidal effect of different doses on the water in question. If the water has previously been filtered, 5 to 10

pounds of hypochlorite per million gallons is generally sufficient. If it is unfiltered or contains abnormally large quantities of organic matter, or iron in a soluble or incompletely oxidized state, more will be necessary, even 20 to 25 pounds in extreme cases. Not only does the presence of oxygen-absorbing impurities in the water necessitate the use of more hypochlorite, but sudden fluctuations in the quantity of these impurities must be guarded against by adding about 25 per cent to the amount of hypochlorite apparently required. It is therefore commonly desirable to add this after such impurities have been removed by filtration.

The hypochlorite is applied to the water as a 1 to 2 per cent solution (a large amount remains in the tank as insoluble sludge, and must be removed and disposed of in some way—an objection to this process). Even this weak solution is very corrosive of wood, steel, copper, and other substances, and all tanks and pipes for handling it should be of concrete, cast-iron or pure wrought iron, lead, acid-proof bronze, vulcanized rubber, or glass.

An odor similar to that of iodoform is sometimes imparted to the water by hypochlorite, probably due to some action on the organic matter in the water, or possibly in some cases to its reaching the consumer before the reactions are completed. To prevent this, care should be taken not to overdose the water because of variation in rate of flow or in amount of oxygen-consuming matter present; the hypochlorite solution should be kept well mixed and tested at intervals to determine its strength, and sufficient time for complete reaction should be given in a storage- or clear-water basin. If in spite of these the taste persists, it can be removed by applying sodium thiosulphate fifteen to thirty minutes after the hypochlorite has been applied, half as much of the former being used as of the latter. The thiosulphate stops the germicidal action of the hypochlorite, and therefore should not be added until several minutes after the latter has been.

Some of the worst cases of taste in chlorinated water have been due to the presence in the water before treatment of phenol

from by-product coke ovens and commercial gas plants. Apparently the only remedy is preventing the discharge of such wastes into water that is to be used for a potable supply.

Chlorine gas has been used in an increasing number of plants since about 1913. The reactions are similar to but simpler than those of hypochlorite and do not require carbonic acid. The gas is practically pure chlorine and therefore theoretically only about one-third as much is needed as of hypochlorite, and practically 17 to 20 per cent may give equivalent results. There is no sludge to dispose of; and practically no labor or oversight is required if the gas be applied by an automatic apparatus that regulates the dose to the amount of water treated; while the use of hypochlorite requires labor and machinery for handling the powder, mixing the solution, and removing the sludge. Also the space required for storage and dosing-apparatus are insignificant. On the other hand, the gas is more expensive than the powder. The gas is furnished compressed into liquid form under a pressure of about 100 pounds and contained in steel cylinders 8 inches in diameter and 5 feet high, which hold 100 pounds of liquid chlorine. As the gas is used the pressure decreases, and consequently the velocity of flow if not controlled in some way; for which reason some apparatus is necessary for automatically securing a uniform rate under varying pressure. One or two very satisfactory types are on the market. Several plants manufacture their own chlorine gas and discharge it into the water without liquefaction and therefore need no rate control apparatus. Most plants that previously used hypochlorite have changed to liquid chlorine since about 1917.

Ultra-violet rays have considerable germicidal effect, but do not penetrate to any considerable depth in water. Consequently the water must be passed by the lamp in thin layers. Also any bacteria embedded in solid matter, or sheltered from the rays by minute particles of such matter, are not affected by the rays; consequently the water must be free from suspended matter. Two or three cities of Europe have used this method, but none as yet (1924) in this country. Cheap electricity for

operating the mercury-vapor lamp that produces the rays is one condition apparently necessary for commercial practicability.

Ozone has been used as a germicide in numerous experimental plants and a few municipal service ones, but the latter have all, the author is informed, proved unsuccessful. Theoretically, ozone should be ideal for this purpose, for it is an active agent in furnishing nascent oxygen and contributes no impurity or taste to the water. It produces some bleaching effect and oxidizes other organic matter as well as bacteria; for which latter reason the presence of any organic matter in the water reduces the germicidal effect, and the water should be fairly free from such matter or any other that absorbs oxygen. The ozone used for water treatment has been obtained by passing a silent electric discharge through air. Ozone is only slightly soluble in water, and the chief difficulty in its use has been in securing the contact of each molecule of water with a bubble of ozone, which is necessary if no bacteria are to escape oxidation. This has generally been attempted by distributing the gas in fine bubbles at the base of a tower through which water is passed.

Copper sulphate was first used for destroying algæ in water-works reservoirs in 1903 by George T. Moore and Karl F. Kellerman, of the U. S. Dept. of Agriculture, and since then has been so used in scores of instances, generally with success. Just how it acts on the algæ is not definitly known, but minute quantities suffice to kill practically all present, although different organisms require different amounts. Too much copper in water would probably be injurious to the human system; but to be so would require vastly more than the amount required for exterminating any of the algæ. Moreover, when dissolved in hard water, the copper solution reacts with calcium bicarbonate and in a few hours forms copper hydrate, which first becomes a colloid and then precipitates as a suspended solid. In soft water the same reactions take place, but more slowly. Organic matter in solution tends to retard the action, but suspended matter hastens the precipitation. Cold retards the action.

It is very important that just enough copper sulphate be

used; for too little will be practically ineffective and too much may kill fish and other animal organisms in the reservoir. Kellerman says that 1.2 pounds per million gallons will kill trout, 2.5 pounds will kill carp or suckers, 3.5 pounds catfish or pickerel, 6—perch, 10—sunfish and 17—black bass. As just stated, the hardness of the water, amount of organic matter, presence or absence of carbonic acid, temperature and kind of algæ to be destroyed, must all be considered. Some algæ require 20 times as much copper as others. The following quantities are given by Kellerman as applying to water at about 60° F., to be increased by about $1\frac{1}{4}$ per cent for each degree below this, and decreased at the same rate for higher temperatures:

Asterionella, 0.8 pound per million gallons of water. Spirogyra, 1.7 pounds. Anabæna, 0.8 pound. Oscillaria, 1.7 pounds. Uroglena, 0.4 pound. Beggiatoa, 41.5 pounds. Crenothrix, 2.5 pounds. Besides Beggiatoa, only one other organism named by Kellerman requires more than 8 pounds. Those just named are the ones that most commonly give trouble in reservoirs, but Kellerman gives figures for 36 different kinds of organisms. In calculating the amount to be used in a reservoir, the bottom, stagnant water in a deep reservoir need not be included in the amount of water to be treated. Organisms sometimes become concentrated just above the stagnant layer, are not reached by the copper, and later are brought to the surface by vertical currents caused by temperature changes or wind, requiring a subsequent treatment.

The destroyed algæ of course decompose more or less rapidly, but their small amount as organic matter makes the effect negligible. The odor resulting from the oils released may, however, be noticeable for a few days. If possible, it is desirable to by-pass the reservoir and not use water from it for a few days during and immediately following the application of copper.

Algæ in a reservoir are found to aid in the destruction of bacteria in the water, and a destruction of algæ is followed by an increase in the number of bacteria, but these are probably harmless varieties.

Iodine treatment for goiter. Rochester, N. Y., in 1923 began the practice of adding an iodine salt to the water supply at the rate of 0.0002 of a gram per gallon, as a preventive of goiter; this being continued for a period of two weeks at six-month intervals.

Art. 21. Applying Chemicals

Application of the chemicals referred to in this chapter is attended with more or less difficulty, in some cases because of their corrosive nature, in others because of the difficulty of dissolving them or keeping them in solution; and in all cases because of the desirability of applying them in quantities having a definite relation to the quantity and quality of the water treated. The storing and handling of them also require attention, for several of them deteriorate on exposure to air, and the quantities involved are often large. One million gallons per day may require, under average conditions, about 350 pounds of alum and 150 pounds of lime and 10 pounds of chloride of lime; the whole amounting to 510 pounds a day, or 93 tons a year. Softening may require 1500 pounds or more of lime. St. Louis, Mo., used 19,515 tons of chemicals in treating 34,690 million gallons of water during the year 1915–1916.

Time is required for the dissolving of the chemicals, and more or less stirring or mixing, and this would require a retention and agitation of the entire mass of water for this purpose if the chemical be added directly to it, which would entail great expense. Therefore, a more or less strong solution of the chemical is generally made, which can be mixed with the water easily and quickly. In some plants, however, alum, hydrated lime, soda ash and ferrous sulphate have been added directly in the powdered form. Copper sulphate is dissolved directly in the reservoir from the lump.

Where practicable, the chemicals are mixed in tanks elevated above the water to be treated, so that they may flow to it by gravity. In the case of each chemical (except liquid chlorine, and except in those plants where the chemical is fed in powdered form), the plant for applying it consists of a

dissolving tank, solution tank, contrivance for regulating the dose, and piping for carrying it to the water to be treated. The dissolving tank generally has a perforated bottom, on which the chemical rests while it is being dissolved by hot water sprayed on it or flowing up through it to an overflow. A calculated amount of chemical is weighed out and placed in the dissolving tank, and the solution of it flows into the solution tank, where more water is added to bring it to the desired dilution. A paddle or paddles are kept constantly revolving in this tank to keep the solution of uniform strength throughout. This tank has at the bottom a drain to remove sludge or insoluble material that may have been contained in the chemical. A pipe generally leads to an orifice box, where the rate of flow is controlled, a valve in this pipe being opened or closed automatically by a float in that box so as to keep a constant head over the orifice; or in some plants this head is varied with the amount of water being treated. When the solution-tank is nearly empty (which should be announced by a low-water alarm) a fresh batch of chemical is dissolved in the dissolving tank.

If quicklime is used, it must be slaked first in an iron tank or trough (in Minneapolis a concrete mixer is used) using about four times as much water as lime, the water being as hot as possible (not falling below 200° during slaking) and being thoroughly mixed with the lime during the fifteen to forty-five minutes required for thorough slaking. When slaked, the lime is placed in the dissolving tank and diluted with four times its weight of water as cold as possible, since slaked lime (calcium hydroxide) is more soluble in cold than in hot water.

Hypochlorite of lime is difficult to dissolve because it is comparatively insoluble and floats upon water. In some plants the drum of hypo is supsended over the dissolving tank, a hole is cut in the top and in the bottom of the drum, and a stream of water under pressure is turned into the top hole, washing the hypo through the bottom hole into the dissolving tank, from which it flows into the solution tank. Or the hypo is first made into a paste in the dissolving tank by use of

mixing paddles and then is flushed slowly into the solution-tank. For small plants Stein recommends using an ice-cream freezer, the can being perforated with numerous holes about

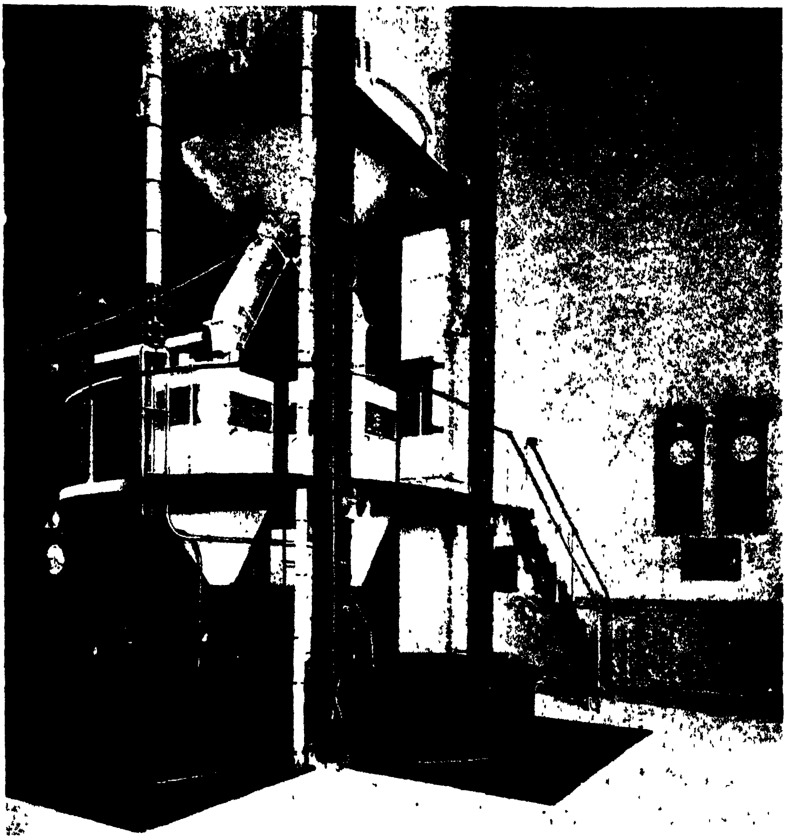


FIG. 9.—Lime-slaking Machinery, St. Louis Filters.

At the top is seen the bottom of a 10-ton "daily supply" hopper. Under the hopper are three scales in which are weighed the supplies for the three slaking tanks, whose tops are seen rising above the floor level. The action of the scales is regulated by a clock, provided in duplicate, by which lime is discharged every one, two or four minutes, as desired; both weighing and dumping being automatic.

$\frac{1}{8}$ inch diameter. A pipe connects the freezer pail with the solution tank. A dose of hypo is placed in the can, the regular paddles are put in place, and hot water is run into the pail at the rate necessary to keep it nearly full, the can being mean-

time turned as in freezing cream. All parts of can and pail should be coated with asphalt paint to prevent corrosion.

Alum, soda ash, and ferrous sulphate once dissolved remain in solution, and the mixing paddles in the solution tank do not need to be kept going after the solution has been mixed. In the case of lime solutions, however, the paddles must be kept going continuously. Hypochlorite must be stirred thoroughly in the solution tank, then allowed to settle and not stirred again, as a large amount of insoluble sludge collects from it, which should not pass out with the effluent, but which must be removed at intervals, for which purpose a waste-pipe in the bottom is desirable.

The amount of chemical solution applied should, of course, vary with the amount of water to which it is applied. Ordinarily the rate at which the purification plant is operated is kept uniform for hours at a time, and the rate of application of chemical can readily be adjusted by hand when the flow of water changes. But in large plants where there are numerous parts to watch, or in small ones where there is apt to be frequent variation of flow through the plant, automatic regulation is desirable. Devices used for this purpose include a weir, venturi meter or other device for measuring the water, and a means by which variations in the flow of this are made to expand or contract an orifice or to vary the head over it.

For feeding dry chemicals, these must be powdered and fed into the flowing water at a positively determinable rate. A common device consists of a funnel in which a screw revolves, which pushes a given amount of powder out of the funnel with each revolution. The number of revolutions per minute thus determines the rate. This powder falls into a stream of water that carries it into the channel through which the water to be treated flows.

In using liquid chlorine, the commercial plant as purchased contains all the contrivances needed, except that its regulator may be checked by placing the chlorine container on a platform scales and noting the amount used from it in an hour or day. Replacing an empty container with a full one and adjust-

ing the regulator to suit the condition of the water are about the only attention needed.

Equipment for manufacturing chlorine gas at the plant consists of a cell wherein chlorine gas is created by passing electric current through brine, which gas passes directly into the water to be treated. The rate at which the gas is created is controlled by varying the amount of current flowing through the brine.

Copper sulphate is applied to a reservoir water by placing the required amount of commercial blue vitriol (calculated for the capacity of the reservoir) in a bag of coarse cloth, a perforated bucket, or wire basket and towing it behind a boat (or two or more containers from outriggers) back and forth across the reservoir in parallel paths about 20 feet apart. Rapid rowing is about the rate of motion desired, although a motor boat may be used. A too-rapid rate is preferable to a too-slow one; for the former simply means covering the reservoir more than once, while the latter may cause a part of the reservoir to receive the entire dose, thus poisoning fish and other animal life. A windy day or anything that promotes circulation of the water aids the distribution of the chemical throughout the water.

CHAPTER V

SOURCES OF SUPPLY

Art. 22. Rain

In the temperate and frigid zones rain (and snow, which is generally included under this term unless otherwise specified) alone is considered as the source of water. For the use of man, whether for domestic or manufacturing purposes or for irrigation, the rain, since it does not fall continuously, must be caught and stored to tide over periods of longer or shorter duration between rainfalls. Nature performs this to a considerable extent through the agency of porous soil and rocks, underground caverns, lakes and ponds, glaciers, and in other ways; man, by the use of cisterns and larger reservoirs, and to a small extent by the storing of ice. The amount of rain that falls directly into a reservoir is generally but a small part of its capacity; but it is that which flows from some drainage area, whether a roof or a watershed of thousands of acres, that gives most of the supply. If this drainage area is the surface of the ground, the run-off or yield is called surface water; but if collected from an artificial surface it is considered solely as rain water. Water is collected in the latter way and stored in cisterns in many low, flat countries or those with very porous soils, where the hungry soil yields none of the rain as surface water, or where there are no natural basins for impounding it. Some Southern cities in our own country and the adjacent countries and islands rely almost wholly upon the collection of water from the roofs of dwellings and other buildings.

Art. 23. Surface-water

By the "yield" or "run-off" of a catchment area, which constitutes surface water, is meant the total amount of water

that flows from a given drainage area, generally in the shape of streams fed by the rainfall upon this area. This is never the whole of such rainfall.

In falling, some rain is intercepted by the foliage and stems of trees and smaller plants, to be later evaporated back again into the air. Of that which reaches the earth, a part flows over the surface and the remainder enters the soil. If the soil be very porous, almost all the rain reaching it may be absorbed; if non-porous, little may enter it. All soil, even the densest rocks, will absorb some water, however. As the unabsorbed water flows over the surface to the lower levels it increases in volume and forms into rivulets, these unite to form larger streams, and the river formed from the union of thousands of such finally enters the ocean. (There are some exceptions, as where streams are wholly evaporated, or where they enter

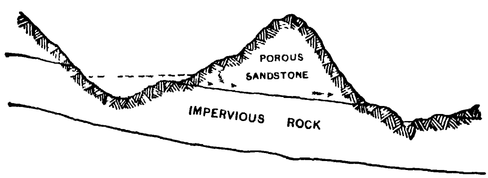


FIG. 10 —Ground Water Diversion by Inclined Strata.

the ground and emerge into the ocean far from the shore and at a considerable depth.)

After a rain has ceased, the streams carry less and less water, but those of any size seldom become entirely dry, even though weeks may elapse between rainfalls and the surface of the ground become dry and dusty. The immediate supply during this time cannot be the rain; but is found to be that portion of previous rainfalls which was absorbed by the earth and which is now being yielded slowly. In general, the more porous the soil the more water it will receive for this purpose during a given time of rainfall; and the finer its grain the more slowly will its supply be yielded and become exhausted. In some instances the ground flow does not reach the same stream as does the surface flow, but is carried by the dip of the strata into another valley, as in Fig. 10.

The ground flow frequently emerges as springs; but the larger part of it ordinarily reaches the stream as a general exuding from the banks and in some cases the bottom of the channel.

A study of the material and dip of the strata, and of the surface conditions—slope, vegetation, existence of ponds, etc., —as well as of the rainfall and other meteorological conditions, is necessary for forming any estimate as to the probable amount and rate of yield of a given watershed, where the actual measurement of this is impossible. Such measurement, to be of value, should be continued for a series of years.

The total amount of rainfall reaching the ground is not yielded by the combined surface and ground flow; a large part is evaporated from the surface and from ponds or other bodies of water, large or small; considerable is taken up by the vegetation, which frequently sends its roots several feet into the soil in search of it, the greater part of this being later evaporated from the foliage; and part is held in the soil by capillary attraction. From many watersheds in the Atlantic and Gulf states the ground water travels hundreds of miles to emerge in the ocean bed far from the shore; as off the coast of South Carolina, east of Matanzas Inlet, Fla., and off the shore of Pensacola, Fla.; which water is withdrawn from the yield of these watersheds.

Art. 24. Rivers and Lakes

The dividing line between surface and river supplies is an indefinite one; but where the supply is taken directly from a river or lake without impounding or storage, this should undoubtedly be called a river or lake supply. The conditions are in many respects similar to those affecting surface waters; but the supply is generally somewhat more constant and of greater volume, owing to the larger drainage area; it is more likely to be polluted, and to lie lower relative to the point of utilization.

Lakes are nature's regulators of flow, and take the place of artificial storage reservoirs, besides contributing to the self-

purification of river waters. They are generally but enlargements of a river channel; although some lakes are formed directly from surface flow or from large springs, and form the sources of rivers; while still others have ground water as both source and outlet. Lakes can in most cases be relied upon as more constant than rivers in both the quantity and the quality of water available.

ART. 25. UNDERGROUND SUPPLIES

The water flowing underground towards a surface stream or the ocean may emerge as springs or be reached by a well dug or bored to and through the porous stratum through which it flows. Such water seldom flows in the form of a stream except as it follows underground channels in limestone rock; but it generally fills the porous stratum throughout, and moves slowly through the interstices towards the outlet. This movement is not always downward, but the outlet must always be lower than the point where the water enters the soil. If, in traveling through such stratum, the ground water reaches a fault, this may be followed to the surface and the water emerge as one or more springs.

ART. 26. OTHER SOURCES

In the form of snow and ice, water is stored by nature and man. Many Swiss streams have as their origin thawing glaciers formed from the snowfall of many years ago. Many thousands of tons of ice are stored annually by man, much of which is used in and for drinking water. But this use is only incidental, and will be considered only in reference to its purity.

Dew and fog have, in a few cases, furnished limited supplies. Construction camps in Mexico have found supplies of fresh water among the sand dunes derived from such sources; and Laguna Honda, San Francisco, among the sand dunes, gives more water towards the city supply than the rainfall on its drainage area would furnish.

Art. 27. Relative Use of Different Sources

The table given herewith shows the relative number of works in the United States which utilize each of the several sources of supply.

The indefiniteness of the classification used by the different contributors of the data makes it difficult to be certain as to the proper placing of some of the plants; and in some cases two or more sources are used by the same city. But the table gives an approximate statement of some interest and value. We see that about half of the plants in the country have ground water (well) supplies, the largest proportion being in the North-central and Northwest sections; and that New England and the Middle states, with their hilly country, contain the largest number of surface supplies. Also that in all the Southern states rivers are used more generally than any other one source, and ground water next.

In the next few chapters the sources above referred to will be considered at length.

Table No. 10

SOURCES OF WATER SUPPLY

Percentage supplies from a given source are of all the supplies in the division in question
Calculated from data for 1916 collected from 800 cities by "Municipal Journal."

Geographical Division.	Wells and Filter Galleries.	Rivers and Creeks.	Impounding Reservoirs and Mountain Streams	Lakes.	Springs.	Per Cent Plants in Division are of Total Number.
New England......	22 1	16 8	31.0	24 8	5.3	14.4
Middle Atlantic....	27 6	19 9	32 6	7 1	12 8	18.0
East North Central.	63 7	15 3	4 7	13 7	2 6	24.3
West North Central..	60.6	22 8	6.1	6.1	4 4	14.5
South Atlantic.... ..	37 5	36 0	15.6	....	10.9	8.2
East South Central..	45 3	23 8	9.5	2 4	19 0	5.4
West South Central..	46 0	24.3	18 9	5 4	5.4	4.7
Mountain.........	16 1	41.9	32.3	....	9.7	4.0
Pacific............	41.2	21.6	27.4	3.9	5 9	6.5
Entire United States.	43 4	21.5	18.1	9.7	7.3	100 0

CHAPTER VI

RAINFALL

ART. 28. QUALITY OF RAIN WATER.

THERE is a popular impression that all rain water is as pure as any natural water obtainable, but this is not borne out by analyses, as was shown in Art. 8. Snow usually removes more impurity from the atmosphere than does rain, owing to its form.

Rain water, on account of its softness, is especially adapted to washing; and where the public supply is hard, roof water collected in cisterns may form a very advantageous auxiliary supply for this purpose. It is also a most wholesome beverage if so collected and stored as to retain its original purity, since the impurities which it contains are not generally injurious unless allowed to accumulate and putrefy. Unfortunately the method of collecting and storing private supplies is often very faulty. Every roof before a rain is foul with excrement of birds, dead insects, leaves, and dust, and in too many cases all of this is washed into the cistern. The first part of each rain should be run to waste, and only after the roof is washed clean should the water enter the cistern. There are many automatic devices for accomplishing this, but few are in common use. A simple two-way valve in a tin or copper breeches-pipe at the bottom of the rain-water leader, to be turned by hand, is a common and effective contrivance if properly used and always so left after a storm as to waste the water until turned.

The best tank for storing water is made of slate. Iron properly coated to prevent rust is excellent also. Particularly in the South, cypress wood is much used for cisterns. Masonry walls are ordinarily used for underground cisterns, but should be absolutely tight, and this condition should be tested at inter-

vals of not more than one year. For a time, at least, water stored in these is apt to be hard owing to the lime in the mortar.

Tanks or cisterns should always be covered to keep out dust and other impurities. They should also be so located and so tightly constructed that contamination from outside is absolutely impossible. Otherwise, their bottoms should be above the top of the sewage in any cesspools located within a radius of several hundred feet; and if privies or other surface

TABLE No. 11.

ANALYSES OF CISTERN WATERS

(Nichols' "Water Supply, Chemical and Sanitary.")

Locality.	Total Solids	Ammonia		Chlorine.	Authority.
		Free	Albu-minoid.		
Boston, Mass. . . .	5 28	0 013	0 008	0 32	W R. Nichols
Same, filtered * . .	6 56	0 012	0 007	0 36	W R. Nichols
Boston, Mass	3 24	0 005	0 011	0 10	W. R Nichols
Same filtered * . .	4 80	0 024	0 016	0 12	W. R. Nichols
Boston, Mass. . . .	3 48	0 021	0 007	0 69	W. R. Nichols
Same, filtered *	5 20	0 007	0 007	0 70	W. R. Nichols
Wilmington, N. C. .	5 05	0 002	0 015	0 70	C. W. Dabney
Wilmington, N. C. . .	6 90	0 016	0 008	0 52	C. W. Dabney
Wilmington, N. C. .	3 60	0 005	0 008	0 52	C. W. Dabney
Cincinnati, O	2 68	0 004	0 123	0 55	C. R. Stuntz
Cincinnati, O	4 72	0 275	0 055	2 76	C. R. Stuntz
Cincinnati, O .	4 48	0 027	0 118	1 97	C. R. Stuntz
Cincinnati, O . . .	7 96	0 004	0 016	trace	C. R. Stuntz
Cincinnati, O	4 10	0 020	0 360	trace	C. R. Stuntz
Watervliet, N Y		0 1050	0 0175	0 200	W. P. Mason

* These cisterns were provided with a brick filtering-wall—the inefficiency of which is evident.

deposits of excreta be located within that distance, the tank should be entirely above ground. The air reaching the tank should be pure; and no overflow or other pipe should connect it directly with the sewer. In New Orleans, where, until a few years ago, a large part of the water supply was from private cisterns, the usual capacity of the dwelling-house cistern was about 2000 gallons. They were raised a few feet from the ground, and their contents protected by a lid or cover. Some

were placed under the shade of a balcony; a few had a special roof over them; but the majority had only such protection from the rays of the sun as was afforded by their position against the house wall. All that were not tightly covered were required to be covered with fine netting to prevent mosquitoes breeding in them. The deposit in New Orleans cisterns Dr. Smart found to collect at an average rate of an inch a year. Sediment collects from all rain water, and should be removed before reaching any considerable amount, since it is stirred up by the inflow from each rain, and the organic matter therein is continually decomposing.

The table on page 107 gives the analyses of a few cistern waters.

ART. 29. QUANTITY OF RAINFALL

Rainfall, being the origin of all supplies, is the basis of calculation of the amount available from whatever source, and a consideration of the amount of rainfall is an essential foundation for further discussion.

The rain which will fall at any one place in any day, month, or year cannot be accurately predicted by any known theory or science; but a record of past rainfalls will afford an aid to our judgment in estimating such amount, and in fact forms practically the only basis for such judgment; although in some cases probable changes in certain large features of the country—such as deforestation—may be considered. (Authorities differ as to whether the presence of forests causes increased precipitation; but there seems to be little proof that such increase, even if there is any, is at all considerable in amount.)

The rainfall of each locality differs from that of every other one, not only in long-term averages, but even in times of drought and flood. Points only a hundred miles apart often have excessive or different rainfall in different years. Even on one watershed (the Esopus, New York's Catskill water supply) the seven-year averages of ten rainfall gaging stations varied from 39.6 to 58.6 inches; and the average of all was 20 per cent higher than the normal as given by the U. S. Weather Bureau; while at

Albany, 50 miles away, the average for the same seven years was 23 per cent below this normal. The rainfall at any one point may at any time be double the average, or only a half or a third of it.

In spite of these wide variations, there are certain general laws that can be learned and upon which plans may be based. If fact, many plans *must* be based upon some rainfall assumptions. With sufficient past rainfall records as a basis, we can safely assume that the rainfall for no year will ever be less than a certain minimum or more than a certain maximum. We can also feel safe in assuming that the total rainfall for a period of thirty years will always fall within certain limits. It is believed that a thirty- or forty-year rainfall record will include the maximum floods and droughts that are likely to occur on that particular area, and also that during that time a complete cycle of rainfall rates will have been completed that will give a fairly correct average.

TABLE NO. 12

SOME EXTREMES OF ANNUAL PRECIPITATION AND THEIR PROPORTION *

Location.	Interval.	Inches of Annual Precipitation.			Per Cent of Maximum to Minimum.
		Average.	Maximum.	Minimum.	
Boise, Idaho	1868–1914	13 81	25 80	6.69	386
Boston, Mass	1818–1914	44 10	67 72	27.20	250
Des Moines, Iowa	1879–1914	32 23	56 81	18 24	311
Detroit, Mich	1871–1914	32 00	47 69	21 06	226
Grand Junction, Colo	1893–1914	8 24	11.61	3.64	320
Little Rock, Ark	1880–1914	49.11	75 54	33.32	230
Los Angeles, Cal	1877–1914	15 71	38 18	5.59	683
Madison, Wis	1869–1914	31.71	52 91	13 49	392
New Orleans, La	1836–1914	56.05	85 73	31.07	276
Omaha, Neb	1871–1914	29.68	48 92	15 49	316
Pueblo, Colo	1889–1914	11.78	18 58	6.14	302
Richmond, Va	1872–1914	41 63	72.02	27.65	260
Rochester, N. Y	1871–1914	33.69	49 89	20 30	245
St. Louis, Mo	1837–1914	39 96	68.83	23 38	294
St. Paul, Minn	1837–1914	27.31	49 69	10 21	486

* By C. P. Birkinbine.

Where no record of such duration is available for the area in question, the only available plan is to use such records of adjacent and surrounding territory as have been secured; especially of such minor areas as are similarly situated with respect to conditions affecting rainfall. When such comparisons are made, local records being wanting, plans based upon them should allow for a considerable "factor of safety." In making such comparisons, where records of several neighboring areas are available each should be weighted with a factor expressing its propinquity to the area in question and its similarity of conditions affecting rainfall; the sum of the products of the several local averages and their respective factors, divided by the sum of the factors, giving the probable average desired.

Rainfall records are now taken by the U. S. Weather Bureau, or by observers recognized by it, at more than 5000 stations in the United States, although comparatively few of these extend back for thirty years. These records for any given state or other area can be obtained from the Bureau for this use, and other uses that will be designated further on.

Reliable rainfall records continuous for more than seventy years have been kept at Boston, New Bedford, Waltham, Lowell, Amherst, Worcester, and Cambridge, Mass.; Providence, R. I.; Albany, Rochester and New York City, N. Y.; Newark, N. J.; Philadelphia, Charleston, New Orleans (except during 1860 to 1868), Cincinnati, St. Louis, Fort Leavenworth and St. Paul.

Instead of comparing inches of rainfall, it is desirable for many purposes to compare percentage variations—that is, the percentages that the individual yearly rainfall rates are of the average rate for each station. For instance, a fair uniformity will often be found, in a given section of the country during twenty or thirty years, in the number of yearly records that exceed the local average by ten, twenty, thirty, etc., per cent. And it is the extent and frequency of such variations, and not the year of their occurrence, that is most important or can be even approximately predicted.

The year is too long a rainfall period for many purposes, and the unnatural division of the records by the calendar year is

objectionable for several reasons. Consequently most records are given by months; and the water year is taken by some as beginning with November 1st or December 1st; or locally at some date well outside of the annual "rainy season." The monthly variations are much greater than the annual, and the monthly records over long periods for several neighboring

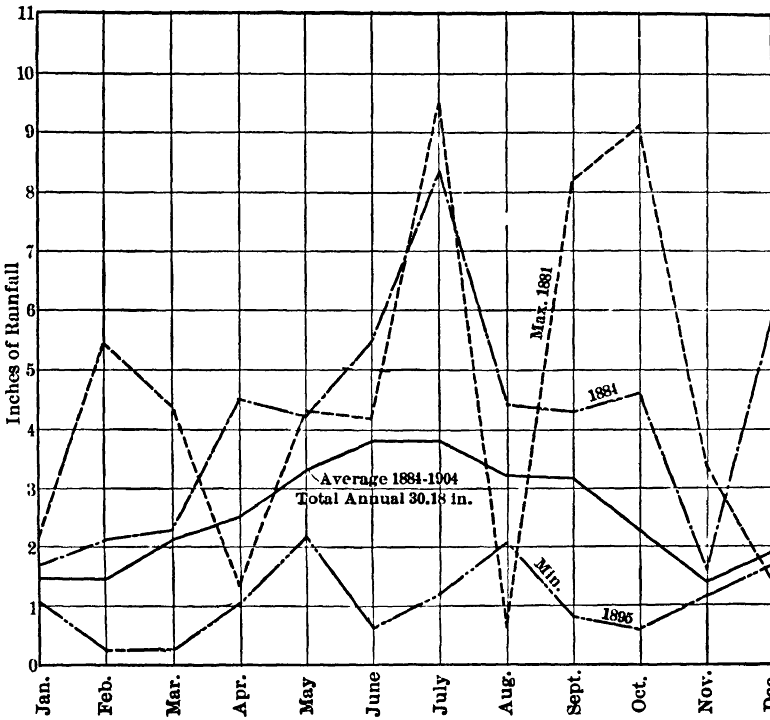


FIG. 11.—Fluctuation of Monthly Rainfall at Madison, Wis.

stations will not often show any striking similarities. But comparisons of percentages of variation from the mean monthly precipitation are very useful. Especially useful are cumulative surplus and deficiency percentages, showing length and intensity of drought and wet seasons. Cumulative diagrams are recommended for this purpose, an ordinate being plotted for each month representing the total precipitation from the beginning of the period considered to the end of the month in question.

If a straight line be plotted on a slope representing average rainfall and starting from the same zero, the two lines will cross at the time when the deficiency or excess has been offset by later

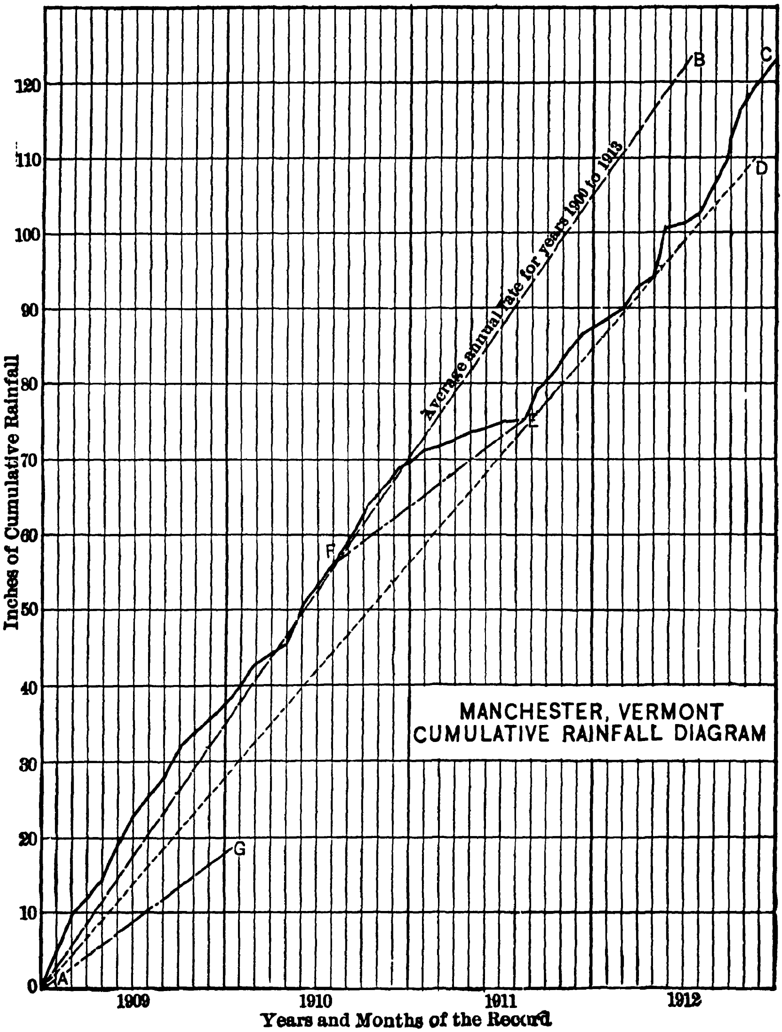


FIG. 12.—Cumulative Rainfall Diagram for Manchester, Vt.

excess or deficiency; and the angle between the two lines at any point will indicate whether the precipitation at that time is less or greater than the average.

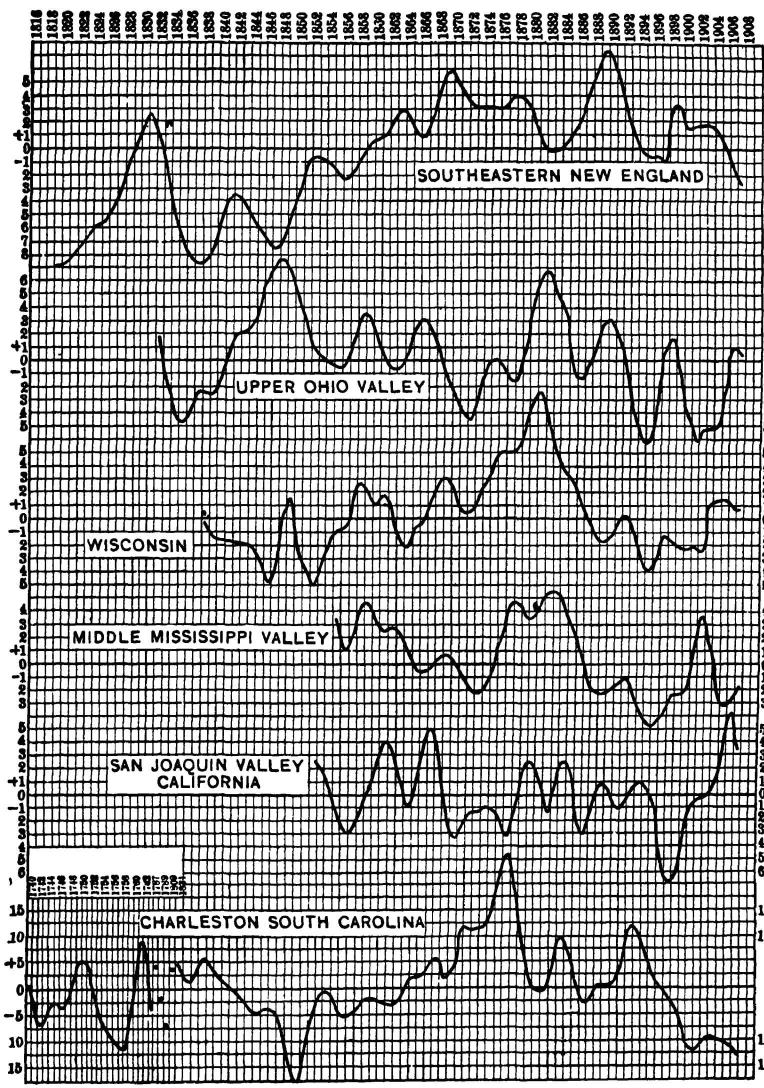


FIG. 13.—Progressive Mean Annual Precipitation at Several Locations in the United States.

From "Flow of Streams and Factors that Modify It," by D. W. Mead, in Bulletin of University of Wisconsin. Blanford's progressive average is used $= \dfrac{a+4b+6c+4d+e}{16}$, in which a, b, c, d and e are the annual precipitation for the year in question (c), the two years preceding and the two following. The zero line in each case is the average for that city.

Table No. 13

RECORDS OF SOME SEVERE STORMS *

Location.	Date.	Depth in Inches.	Duration.		Average Rate per Hr.
			Hrs.	Min.	
Albany, N. Y.	July 10, 1876	1.12	0	10	6 72
Alpena, Mich	September 20, 1884	1.05	0	11	5.73
Amanda, Ia.	July 31, 1878	1.56	0	15	6.24
Atlanta, Ga.	April 24, 1889	1.12	0	10	6.72
Atlanta, Ga.	July 23, 1898	4 30	0	51	5.06
Berne, Ind.	August 16, 1913	2.55	0	30	5 10
Biscayne, Fla	March 28, 1874	4 10	0	30	8.20
Brandywine Hundred, Pa	August 5, 1843	10 00	2	0	5 00
Cambridge, Ohio	July 16, 1914	7.09	1	30	4.73
Catskill, N. Y.	July 26, 1819	18 00	7	30	2 40
Chicago, Ill.	May 25, 1896	1.24	0	15	4 96
Collinsville, Ind.	May 23, 1888	1.70	0	12	8 50
Concord, Pa	August 5, 1843	16.00	3	0	5 33
Embarrass, Wis	May 28, 1881	2 30	0	15	9.20
Flatbush, L. I.	August 22, 1843	9 00	8	0	1.12
Ft. Leavenworth, Kan	July 21, 1887	1 90	0	20	5 70
Ft. McPherson, Neb	May 27, 1868	1.50	0	5	18 00
Ft. Randall, S. D	May 28, 1873	1 56	0	15	6 24
Ft. Scott, Kan	October 2, 1881	1 80	0	20	5 40
Galveston, Tex	June 4, 1871	3 95	0	14	16 93
Huron, S. D	July 26, 1885	1.30	0	10	7.80
Indianapolis, Ind.	July 12, 1876	2 40	0	25	5 76
Jewell, Md	July 27, 1897	14.75	18	0	0 82
Lebanon, Pa.	July 10, 1914	5 00	2	0	2 50
Long Branch, N. J	July 14, 1912	0.80	0	5	9 60
Newton, Pa.	August 5, 1843	5 50	0	40	8 25
Newton, Pa.	August 5, 1843	13.00	3	0	4 33
New York, N. Y	May 22, 1881	1.15	0	10	6.90
New York, N. Y	November 18, 1886	0.25	0	2	7 50
Osage, Ia.	August 26, 1881	1.40	0	15	5.60
Ottawa, Ohio	August 21, 1913	2 28	0	40	3 42
Palmetto, Nev	August 7, 1890	8 80	1	0	8 80
Paterson, N. J	July 13, 1880	1 50	0	8	11 25
Philadelphia, Pa.	September 12, 1862	9 00	5	0	1.80
Portsmouth, Ohio	June 22, 1851	1 75	0	15	7 00
Sandusky, Ohio	July 11, 1879	2 25	0	15	9 00
San Francisco, Cal.	December 20–21, 1866	7 76	8	45	0 89
St. Louis, Mo.	August 15, 1848	5 05	1	0	5 05
Stroudsburg, Pa.	August 1, 1913	7 50	3	50	1 96
Tridelphia, W. Va	July 19, 1888	6 90	0	55	7.53
Washington, D. C	July 26, 1885	0 96	0	6	9.60
Worthington, Minn.	August 20, 1913	8 00	11	0	0.73

* By C. P. Birkinbine.

For many purposes, such as designing spillways, the month is much too long a unit, and single storms, hours or five-minute periods are taken. For these, self-recording rain gages are necessary for measuring the precipitation, and five-minutes is generally taken as the unit of time. The important fact to be learned is the maximum amount of rainfall to be expected in any five, ten or fifteen-minute and longer period; and the total rainfall in this particular storm that has preceded this maximum. Here the records of no one station, not even of one on the area

TABLE No. 14

PERCENTAGES EXTREME INDIVIDUAL ANNUAL RAINFALLS ARE OF MEAN ANNUAL RAINFALL RATES

Location.	Period, Years	Per Cent of Mean.	
		Maximum.	Minimum.
Boston, Mass	96	153	62
Detroit, Mich	43	149	66
Little Rock, Ark	34	154	68
New Orleans, La	78	153	55
Omaha, Neb	43	165	52
Richmond, Va	42	173	66
Rochester, N. Y	43	148	60
St. Louis, Mo	77	172	59
St. Paul, Minn	77	182	37
Brooklyn, N. Y	87	135	73

in question, should be considered sufficient, for extreme rainfall rates may visit any given locality only once in a century; but that once, washing out a dam, may cause great damage and loss of life and should be provided against. For this reason maximum rainfall rates for all sections at all similarly situated are used. Scores of such rates have been collected and several formulas have been evolved for expressing such maximum rates in terms of duration. Fig. 14 shows several of these which are perhaps as reliable as any for general use. Talbot's maximum is supposed to include all storms for sections east of the Rocky Mountains, although a few storms (indicated by ×'s on the diagram) have exceeded it. Sherman's maximum for

Boston will probably apply to the eastern United States, but may be exceeded at any place once in ten years or so. The two "ordinary" curves represent ordinary storms that may come almost any year. The curve for San Francisco, based on local records, indicates the less intensity of storm precipitation on the middle and southern Pacific coast. Some maximum figures are given in the table on page 114.

TABLE No. 15

ANNUAL RATES AND SUCCESSIVE AVERAGES.—ST. PAUL

Year.	Rainfall.	Average to Date.	Year.	Rainfall.	Average to Date. Forward.	Average to Date. Backward.	Year.	Rainfall.	Average to Date.† Forward.	Average to Date.† Backward.
1837	24.1	....	1862	34.5	26.3	28.6	1887	25.9	27.4	28.4
1838	27.7	25.9	1863	15.7	25.9	28.5	1888	25.8	27.4	28.5
1839	21.2	24.3	1864	14.9	25.5	28.8	1889	17.1	27.2	28.6
1840	23.1	24.0	1865	38.1	25.9	29.1	1890	23.5	27.1	29.2
1841	21.6	23.5	1866	27.9	26.0	28.9	1891	21.8	27.0	29.5
1842	26.7	24.1	1867	33.6	26.3	28.9	1892	32.6	27.1	29.9
1843	23.8	24.0	1868	30.7	26.4	28.8	1893	26.0	27.1	29.7
1844	30.4	24.8	1869	32.2	26.6	28.8	1894	25.8	27.1	29.9
1845	25.5	24.9	1870	32.1	26.7	28.7	1895	24.3	27.0	30.2
1846	26.0	25.0	1871	30.7	26.9	28.6	1896	34.7	27.1	30.6
1847	21.9	24.7	1872	29.6	26.9	28.5	1897	30.5	27.2	30.3
1848	23.2	24.6	1873	34.6	27.1	28.5	1898	25.3	27.2	30.3
1849	49.8	26.5	1874	35.5	27.4	28.4	1899	27.5	27.2	30.7
1850	25.5	26.4	1875	30.7	27.4	28.2	1900	34.2	27.3	31.1
1851	23.4	26.2	1876	23.6	27.3	28.1	1901	25.8	27.3	30.7
1852	15.0	25.6	1877	28.7	27.4	28.2	1902	31.8	27.3	31.3
1853	20.4	25.3	1878	22.6	27.3	28.2	1903	37.9	27.5	31.2
1854	26.5	25.3	1879	32.5	27.4	28.4	1904	33.8	27.6	30.2
1855	24.9	25.3	1880	29.8	27.4	28.2	1905	30.8	27.6	29.7
1856	22.7	25.2	1881	39.2	27.7	28.2	1906	33.2	27.7	29.5
1857	32.1	25.5	1882	23.1	27.6	27.8	1907	23.1	27.6	28.5
1858	20.1*	25.2	1883	26.5	27.6	27.9	1908	31.6	27.7	30.3
1859	29.1	25.4	1884	26.1	27.6	28.0	1909	31.8	27.7	29.7
1860	34.2	25.8	1885	25.3	27.5	28.1	1910	27.6	27.7	
1861	30.5	26.0	1886	22.9	27.4	28.2				

* Estimated.

† The averages in the last column are obtained by beginning with 1910 and working backward. This gives the averages for any number of recent years. It is believed that most records prior to 1880 or 1870 are too low, and 28.3± would probably be a more correct average for St. Paul than 27.7.

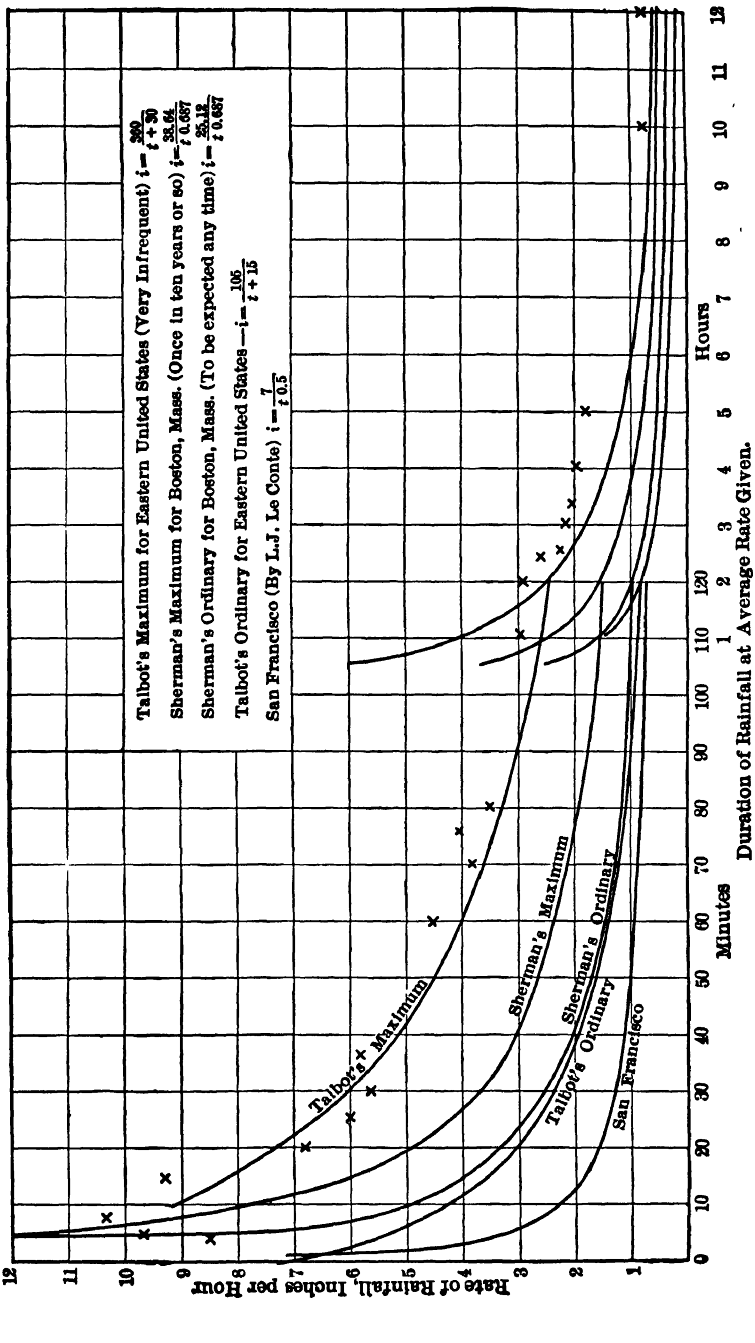


FIG. 14.—Rainfall Curves.

Even in arid regions quite high rates may be experienced. At Yuma, Ariz., there fell in one day in 1899 0.5 inch, and the total for the entire year was 0.6 inch. In 1909 4.01 inches fell in one day and 8.63 inches in the entire year. But the highest rates referred to above probably never occur in the dry regions between the Rockies and the Pacific Coast mountains.

Art. 30. Conditions Affecting Rainfall

Rainfall cannot be affected by any changes in the earth's surface made by men. Few, if any, authorities now believe that the destruction or planting of forests has any appreciable effect in causing precipitation; and the effect of either upon the total amount of moisture evaporated for later precipitation is negligible.

In all of this discussion on precipitation, snow and hail are included as rain. In many localities the relative amount of snow as compared to the rainfall is small. (One foot of fresh snow fall is equivalent to between 1 and 2 inches of rain.) On the other hand, in other localities it is the most important form of precipitation, this being especially true in mountain regions. At Summit, Cal., 86 per cent of the precipitation occurs as snow, 298 inches having fallen in one month. Snow in the Utah mountains has been found by measurement to average 48 inches depth, containing 25 per cent water. This high water content is due to packing under the weight of superimposed snow. (After falling, snow may pack by melting and weight and absorb moisture so as to be equivalent to 50 per cent its depth of water.) Since the water content of snow varies, the only reliable method of recording it is as the depth of equivalent water (rain), and this is the practice.

While the annual precipitation on any given area varies considerably, its average rate and the variations therefrom are approximately fixed within certain limits; and they vary with certain conditions. The conditions exerting most influence on rainfall are location with reference to an ocean or other large body of water, and with reference to the path of prevailing

winds from such body of water; whether the area is in or near the ordinary track of storms; location relative to mountain ranges, especially high ones at right angles to the prevailing rain-bearing winds; and elevation above sea level.

Rain is the precipitation of moisture that has been evaporated by warm air, caused by a chilling of the moisture-laden air. Swampy lands contribute considerable moisture, but their area is so small compared to that of the oceans that the latter are the chief controlling source of moisture. In some localities the evaporated moisture travels but a short distance before it is precipitated; but over most of the United States the precipitation is of moisture evaporated from the Atlantic or Pacific ocean, the Gulf of Mexico, and to a small extent from the Great Lakes.

The moisture-laden air may travel low, in which case it is precipitated on the slopes of the mountains near the ocean; or it may travel sufficiently high to pass over these and be precipitated further inland. The higher the mountain the greater the probability that it will receive a large share of the moisture. But atmospheric disturbances that chill the moisture-laden air may occur anywhere, when the precipitation will be more or less in proportion to the amount of moisture carried by the air. Therefore the nearer an area is to an ocean, other things being equal, the greater its rainfall. But this is modified by the direction of the prevailing winds. If, or while, these blow from land to ocean there can be little rainfall. The Middle Atlantic coast region, where rain is brought from either north, east or south, has a quite uniform rainfall; while Nevada, shut off from the Pacific by the high coast range, may have but one or two rainfalls a year; and in Southern California, though it is near the ocean, the prevailing winds prevent more than occasional rain. The atmospheric disturbances that cause rainfall are most often, in this country, due to the passing over the area of a "storm center"; and the paths followed by most storm centers are more or less definitely fixed. The frequency and intensity of rainfalls therefore vary with the location of the area in relation to the paths of storms.

TABLE No. 16

MEAN ANNUAL PRECIPITATION, BY DISTRICTS AND ALTITUDES

Altitude.	New England.	Middle Atlantic.	South Atlantic.	East Gulf.	West Gulf.	Ohio Valley and Tennessee.	Lower Lake Region.	Upper Lake Region.	North Dakota.	Upper Mississippi Valley.	Missouri Valley.	Northern Slope.	Middle Slope.	Southern Slope.	Southern Plateau.	Northern Plateau.	North Pacific Coast.	Middle Pacific Coast.	South Pacific Coast.
0– 100	43.0	45.1	60.3	59.6	39 4	....	...	...	...	...	...	...	...	...	...	...	...	33.3	10.5
100– 200	45.1	41.9	48.3	....	....	....	...	...	...	...	...	...	...	...	3.0	...	49.9	33.3	
200– 400	....	44.4	55.5	54.2	51.1	53.3	35.0	...	...	42.8	...	...	...	...	...	...	44.6	23.7	15.3
400– 600	....	....	....	....	45.6	47.9	34 8	...	...	36.5	...	...	...	...	...	...	35.2	26.1	
600– 800	....	42.8	....	....	29.7	48 5	36.1	32.3	...	36.4									
800–1000	35.7	37.3	51.9	....	....	40 5	...	34.8	23.8	30.3	36.4								
1000–1500	....	....	....	52 0	....	51.0	...	...	...	...	33.4	...	29.4	...	7.2	16.8			
1500–2000	....	....	....	....	....	....	...	...	16.5	...	15.8	...	...	25.0	...	18.2			
2000–2500	....	....	....	....	....	....	...	...	...	...	...	13.4							
2500–3000	....	....	....	....	....	....	...	...	...	...	...	18.3	19.8						
3000–3500	....	....	....	....	....	....	...	...	...	...	...	16.7	...	...	...	15.1			
3500–4000	....	....	....	....	....	....	...	...	...	...	...	...	...	18.2	9.3				
4000–5000	....	....	....	....	....	....	...	...	...	...	...	13.2	12.1	...	...	15.3			
5000–6000	....	....	....	....	....	....	...	...	...	...	...	13.4	14.5	...	14.3				
6000–7000	....	....	....	....	....	....	...	...	...	...	...	12.2							

There is difference of opinion as to whether elevation affects precipitation. Certainly in some cases where other conditions are apparently equal, precipitation increases with elevation, but there are other locations where no such relation seems to exist. The mean annual precipitation, by districts and altitudes, of several large areas is shown in the table on page 120. These show no apparent consistent relation between altitude and rainfall; but the areas are so large that distance, and topographical features also, undoubtedly exert influences that would hide the relation, if any exists. On the other hand, such a relation appears to exist on the rapid slopes of the mountains in California and Oregon; and eighteen-year records in Arizona, shown in Table No. 17, seem to indicate such a relation.

TABLE NO. 17

AVERAGE FOR 18 YEARS OF JULY RAINFALL AT DIFFERENT ALTITUDES IN ARIZONA

No. of Stations.	Elevation.	Average Rainfall.	Average Number Rainy Days.
	Feet	Inches	
8	Below 1000	0 33	1
16	1000 to 2000	0 63	5
11	2000 to 3000	2 72	10
16	3000 to 4000	2.79	10
20	4000 to 5000	3.95	14
16	5000 to 6000	4 05	12
10	6000 to 7000	5 01	17
5	7000 to 8000	5 70	21
2	8000 or above	7.12	29

	Elevation, Ft.	Average annual Precipitation, In.
At Sacramento, Cal..................	30	19.80
At Folson, Cal..........................	182	23.33
At Auburn, Cal..........................	1360	32.55
At Colfax, Cal..........................	2421	44.01
At Alta, Cal..............................	3612	42.13
At Emigrant Camp, Cal................	5230	50.77
At Cisco, Cal...........................	5939	57 41
At Summit, Cal.........................	7017	47 93

The diagram below illustrates the relation between total precipitation and that from thunderstorms; consequently the effect upon precipitation of location with reference to one of the paths commonly followed by storms.

The above is a very brief statement, and a very general one subject to many minor modifications, of the conditions affecting rainfall. Because of the variations in these conditions, average annual precipitation in different sections of the United States

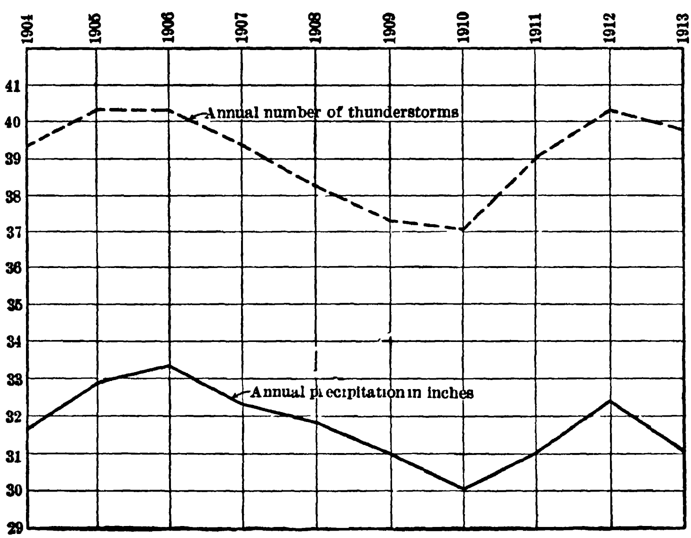


Fig. 15.—Relation between Number of Thunderstorms and Annual Precipitation. By C. P. Birkinbine. Averaged for 127 widely scattered stations in the United States.

varies from 125 inches to 3 inches; while individual yearly records vary from 162.5 inches (Glenora, Ore., 1899) to 0.1 inch at Mohawk, Ariz., in 1895, and nothing at all in Bagdad, Cal., in 1913. But certain areas may be outlined within each of which the conditions are so nearly similar that the average annual precipitations at all points do not differ by more than 10 inches. Such a division, prepared by the Weather Bureau, is shown on the accompanying map.

A similar subdivision by political divisions (groups of states) and consequently less reliable, is shown in Table No. 18.

TABLE No. 18.

MEAN ANNUAL PRECIPITATION IN THE UNITED STATES

(Mean, Maximum, and Minimum Averages of Stations in each District)

Districts.	New England.	Middle Atlantic States.	South Atlantic States.	Florida Peninsula.	East Gulf States.	West Gulf States.	Ohio Valley and Tennessee.	Lower Lake Region.	Upper Lake Region.	North Dakota.	Upper Mississippi Valley
Mean	43 46	43 75	54 09	49 78	54 25	43 15	45.45	35.45	32 61	18 95	34 21
Maximum	47 51	52.34	66 41	57 98	62 61	53.63	51 97	41 28	35 08	23.77	42.83
Minimum .	35 74	37 89	47 55	38 46	51 97	29.70	36.68	30.95	29.53	14 70	27.21

Districts.	Missouri Valley.	Northern Slope.	Middle Slope	Southern Slope.	Southern Plateau.	Middle Plateau.	Northern Plateau.	North Pacific Coast Region.	Middle Pacific Coast Region.	Southern Pacific Coast Region.
Mean	30 25	14 30	22 33	21 61	8 44	12 22	16 36	45 27	29 81	14.59
Maximum	39 93	18 27	33 29	25 02	14 25	16 19	18.25	62 27	45.83	21.52
Minimum.	15.77	12 20	12 11	18 19	2 97	8 48	15 15	35 16	20.87	9.00

From the above it is seen that the warm, moist winds of the North Pacific are robbed of a large part of their moisture by the west slope of the Sierra Nevada and Cascade Range, leaving little for the plateau to their east. The winds blowing over the Gulf Stream into the South Atlantic and Gulf states yield their moisture to them; and as they ascend the valley of the Mississippi and its tributaries their moisture and the consequent rainfall decrease. Above Cape Hatteras the departure of the Gulf Stream from the coast causes its influence to be less felt in precipitation, so that the winds from the Gulf, after passing the low countries of Louisiana, Mississippi, and Alabama, contribute more rain to the Ohio valley than do the winds from the Atlantic to the North Atlantic states.

These figures and presentation of conditions affecting rainfall are given to assist in estimating the probable rainfall at any given place. As stated before, such estimate, where no actual measurements have been taken for a number of years,

must generally be based upon records for other places; and each such record should be given a weight corresponding to the agreement between its rain-causing conditions and those of the area in question. For instance, if the distance from the sea and from the nearest mountains is practically the same for each place, and both lie within or wholly without a path of thunder storms, the precipiation will probably be the same. In general the greatest proportional differences will be found in the districts with the lowest rainfalls. But it should be realized that so little is really known of the influencing conditions, and the variations from year to year are so considerable, that a thirty- or forty-year record taken at the place in question furnishes the only reliable figures, and any other estimates must be considered as subject to an error the amount of which must be estimated by the engineer's best judgment. An example of the variation between rainfall at places similarly situated and near together is furnished by the gages in the Catskill mountains in New York. On the Esopus watershed of 255 square miles, two gages only a few miles apart gave seven-year averages differing by 50 per cent; while various gages throughout the mountains gave averages between 1906 and 1913 ranging from 32.1 inches to 59.0 inches.

Art. 31. Cycles of Low Rainfall

Almost all problems involving precipitation with which the engineer is concerned are affected by extreme conditions rather than long-time averages. Intense rainfall must be provided for in designing spillways and other provisions for handling surplus water. If there were no storage, either natural or artificial, a water supply would be available only during rains; but some storage is always provided, either in the ground or in artificial reservoirs. If these have sufficient capacity only to tide over one dry month, then two consecutive dry months would find the supply entirely wanting during the second. To utilize the entire rainfall (or so much as runs off) for a long series of years would require a reservoir of a capacity equal to the total deficiency in rainfall for the longest period during which the rainfall was below

the average; and to utilize all the run-off from an average rainfall of any less amount would require storage equal to the maximum total continuous deficiency below that average. Boston has experienced twenty-six years of continuous deficiency below the full average, the total deficiency being about 115 inches, or $2\frac{1}{3}$ times the average annual precipitation. This was followed by thirty-five years above the average, except for two years that were only slightly below it. Recently that section has experienced ten years of precipitation continuously below the average. Fig. 13 shows the annual variations from the average at six localities. The dryest year in a century will, for most sections except the most arid, generally have a rainfall between 40 and 60 per cent of the average. The percentage of the average for the dryest one, two, three, four and five years recorded, for three cities, is shown in the table on page 126.

Shorter dry seasons will reach much greater departures from the average. In several sections the precipitation may be almost or quite nothing for one or several successive months almost every year; and in any

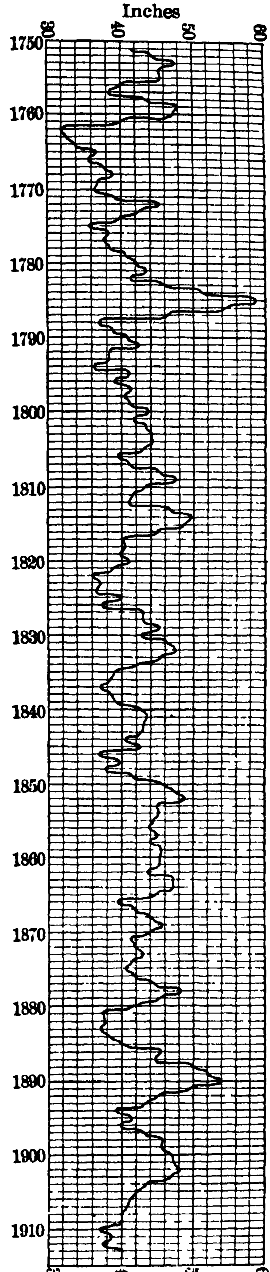


FIG. 17.—Diagram Showing Progressive Average Rainfall in Cambridge, Mass.

The progressive average is made by plotting the successive averages of the rainfall for three-year periods upon the ordinate representing the last year of the period. By X. H. Goodnough; Journal N.E.W.W. Ass'n, 1915.

section it may occasionally be so. Atlanta, Ga., had but 0.1 inch rainfall during fifty-eight days. New York had 0.2 inch rain in forty-six days in 1914. Boston had 0.1 inch in thirty-two days in 1914. Portland Ore., had seventy-five days without measurable precipitation in 1914. At Philadelphia and Boston the rainfall has been less than 1 inch during each month in some year.

DRY CYCLES OF FROM TWO TO FIVE YEARS' DURATION

City.	Length of Record, Years.	Mean Precipitation, Ins.	Dryest Year, Per Cent.	Dryest 2 Years, Per Cent	Dryest 3 Years, Per Cent.	Dryest 4 Years, Per Cent.	Dryest 5 Years, Per Cent.	Dryest 25 Yrs., Per Cent.
Boston...	90	45 3	58	68	76¼	76½	79½	90
Philadelphia	84	42 6	68	73½	77⅓	78	79½	
Denver.....	35	14 3	65	71	77	80	85½	

In the majority of cases, but not always, months of heavy rainfall precede and follow severe droughts; and in the majority of cases the average of any three consecutive months, including the minimum, is at least two-thirds of the monthly average for that year.

Table No. 19

TYPICAL DRY PERIODS

PHILADELPHIA RECORD.

	Dec	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Year.
1841–1842	5 92	1 36	4 27	2.84	5 31	5 87	3 19	11.81	3 79	1 27	1 72	3 49	50 84
1842–1843	3 66	1 44	2 54	4 42	4 72	2 05	1.69	4 54	9 26	4 86	3 22	4 18	46.58
1879–1880	4.69	1 51	2 43	3 53	2 43	0 54	1.67	7.74	5.09	1.10	1 74	1 75	34 22
1880–1881	4 05	3 66	4 76	3 83	0 61	2 71	3 87	0.96	1 18	0.94	3 04	2 02	31.63

NEW YORK RECORD.

	Dec	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Year.
1841–1842	2 70	1 07	2 85	1.25	3 60	3 60	3 30	3 80	2 81	2.10	4 30	1.80	33 18
1842–1843	3 50	1.00	2.31	2 13	2 14	1 00	0 76	1 64	15 26	3 06	5 91	2.82	41.53
1879–1880	4 94	2 02	2 12	4 66	2.90	0 62	1.14	8 53	5 26	1 85	2 81	2.46	39.31
1880–1881	2 27	4 80	4 93	5 81	0.95	3 20	5.35	1.25	0 86	0 97	1.60	2.36	34.35

TABLE No. 20

MEAN MONTHLY PRECIPITATION, BY PRECIPITATION DISTRICTS,* AND PERCENTAGES OF MAXIMUM RANGE OF VARIATION

District Number.	Jan.		Feb.		Mar.		April.		May.		June.		July.	
	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.
1	3 9	69	3 7	51	3.9	59	3 6	55	3 8	52	3.7	41	3.8	66
2	4 3	51	3 8	56	4.6	74	3 0	93	4 0	47	5.5	38	6.2	58
3	1 9	100	2.0	95	2.2	73	2 7	70	3 9	56	4.1	39	3 5	49
4	0 8	137	0 7	171	1 4	64	1.8	61	2 8	89	3 0	73	2 3	70
5	2 0	45	1 5	40	1 6	50	1.4	57	1 4	57	1 0	80	0 4	75
6	0 5	20	0 8	50	0 5	40	0 3	167	0 4	100†	0 3	200†	1 2	138
7	8 1	57	6.0	50	5 4	68	4 3	72	2.7	70	2 3	77	0 8	50
8	5 5	53	4.1	49	4 0	62	2 4	75	1 6	88	0 5	160	0.1	200

District Number.	Aug.		Sept.		Oct.		Nov.		Dec.		Year.	
	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.	Rate.	Variation.
1	4 0	52	3 5	77	3 1	51	3 7	49	3 4	62	44 0	−32
2	6 4	47	5 4	80	3 9	136	3 3	58	3 5	54	53 4	−28
3	3 1	45	3 0	67	2 6	73	2.3	83	2 0	75	33 0	−52
4	2 0	80	1 5	93	1 3	77	0.7	171	0.7	243	18 6	−35 +79
5	0 3	166	1 0	100	1 1	63	1.2	92	1 9	32	14 6	−42
6	1.2	83	0 0	100 †	0.6	67	0.5	60	1 0	100	8 4	−65
7	1.0	87	3 2	128	5 3	92	7.5	99	8 9	84	45 3	−22 +38
8	0 0		0 7	86	1.7	47	3 0	33	5 6	46	29 8	−38 +54

* Precipitation districts referred to are as follows: 1. The North Atlantic, Ohio, and West Gulf; 2. South Atlantic and Gulf; 3. Upper Mississippi and Great Lakes, 4. Eastern Slope; 5. Northern and Middle Plateau; 6. Southern Plateau; 7. North Pacific; 8. Middle Pacific.

†Never any rain during these months in certain localities.

ART. 32. GAGING RAINFALL

Measurements of snow are ordinarily recorded in inches of fall as found upon a level surface free from drifts; but it is very difficult to obtain an average depth in windy weather, although the best judgment must be used to ascertain this. Generally, besides expressing this in inches, a cylinder of snow of this depth is collected and melted in a can or tube of the

same diameter as the cylinder of snow, and the depth of water resulting is recorded as precipitation or rainfall.

Measurements of rainfall are taken by rain gages. The U. S. Weather Bureau standard consists of a circular cup of thick brass, its top brought to a chisel-edge, the bottom cone-shaped and connected with a deep tube of known diameter into which the rain flows from the cup. (See Fig. 18.) The area of the top of the cup and that of the tube bear a known relation to each other—10 to 1 is a convenient ratio—and the depth in the tube is measured by a stick so graduated that, when it is lowered to the bottom of the tube, the scale will give the actual depth of rainfall, allowance being made in the scale for both the relative areas and the displacement of the stick.

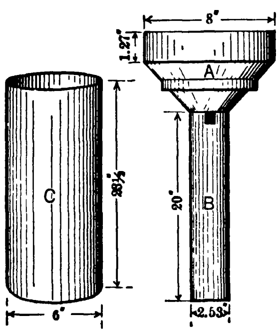


FIG. 18.—Rain-gage.

The depth is customarily expressed in inches and decimals of an inch. The readings are taken daily and at the beginning and ending of each storm.

For many purposes it is desirable to know the rate of fall for short intervals of five minutes or less, and for ascertaining this self-recording gages are necessary. Several styles of such gage have been used. One is the "tipping-tank," which tips and empties itself as soon as it has received .01 inch of rainfall, immediately returning to an upright position, the time of each discharge being automatically recorded. Another gage consists of a tank suspended by a spring-balance, a pencil attached to the tank continuously recording its vertical position upon a cylinder revolved by clockwork once in twenty-four hours. In using any recording gage the total water caught should be retained, and measured or weighed each day as a check upon the record. Automatic recording gages should be used only where they can be inspected frequently, as they easily get out of order or freeze.

The size of the collector-cup seems to have some effect

upon the catchment. For example, of four 3-inch cups and one 8-inch one, in use on Mt. Washington, the average total amount collected by the 3-inch cups in one year was 46.26 inches, while that recorded by the 8-inch cup was 58.70 inches. It is probable that the larger the collector-cup the more accurate the result. It is maintained by many that gages at the surface give less accurate results, since they receive not only the actual precipitation, but also a certain amount of moisture from the surrounding ground which, after falling, again rises by splashing and evaporation and is reprecipitated. A large number of the gages of the signal service are placed upon the roofs of tall buildings, and in cities this is generally necessary but leads to inaccuracies owing to the effect of buildings upon the direction and intensity of the wind; but in open country a height of 3 to 6 feet from the surface will probably give the most accurate results. The gage should be at least as far from any building or other obstacle as the top of this is above the gage. The rim of the collector-cup should be level.

CHAPTER VII

SURFACE WATERS

Art. 33. Evaporation

If all the precipitation upon a given area reached the streams draining it as run-off, the calculation of the amount of the latter would be simply finding the product of the total area of watershed by depth of precipitation. But, as has been stated, much of the precipitation returns to the air by evaporation. Since, however, all but a very minute portion of the rainfall on a given watershed that does not leave it as evaporation leaves it ultimately as yield, the latter could be obtained by taking the difference between the total precipitation and the total evaporation, if these two could be determined.

The amount of direct evaporation depends upon the degree of dryness of the air, the temperature of the air and of the soil or water from which the evaporation takes place, upon the amount of moisture in the soil, and upon the force of the wind. Most measurements of evaporation have been made from water surfaces or by an evaporometer. Water surfaces form but a small proportion of the total areas of most catchment basins, but the amount of evaporation from these and from the reservoirs themselves can be ascertained much more accurately than that from earth and vegetation. The tables No. 21 and 22 give the evaporation from water surfaces at a number of places in this country.

Of the Sweetwater data, those for the first four years were gaged in a pan floating in the reservoir; for the last three years by a Piche evaporometer. Both this and the Richards evaporation gage are thought by some to give inaccurate results, since they are affected by the temperature of the air only, which is seldom the same for any length of time as that of a near

body of water. A more accurate method would seem to be to measure the actual loss from a pan filled with water and floating in a lake or other body of water. Such a pan and scale are shown in Fig. 19. Owing to the protection from wind offered by the sides of the pan, this may give results slightly less than the evaporation from lake or pond surfaces.

Table No. 21

EVAPORATION FROM WATER SURFACES, IN INCHES

Location.	Monthly.			Annual.			Mean Annual Rainfall.
	Max.	Min.	Mean.	Max.	Min.	Mean.	
Boston, Mass.	7 50	0.66	3.29	43.63	34.05	39.20	45.3
Sweetwater, Cal.	9 02	0 25	4 51	58.65	48.68	53.88	
Rochester, N. Y.	6.20	1 51	2 61	34.4	30 0	31.3	33.7
Middle Atlantic States	...	...	....	48.1	25 2	39.9	43.7
South Atlantic States	..	.	. .	51.6	38.4	45.3	51.1
East Gulf States		.	.	56 6	45.4	50 6	54 2
West Gulf States	.	. . .	. .	52 4	45.6	48 9	43.1
Ohio Valley and Tennessee	..	. . .	. .	54.8	44 5	49 4	45 4
Lower Lake	..	. . .	...	38 6	32 9	35.8	35 4
Upper Lake	..	....	.	36.8	23 0	27.7	32 6
Upper Mississippi		. .	. . .	52 2	28 1	38.8	34.2
Extreme Northwest			.	31 0	22.1	26.7	45.3
Yuma, Ariz.		.	. .	. .	..	95.7	
San Diego, Cal.			..		. .	37 5	

Table No. 22

EVAPORATION FROM WATER SURFACES, BY MONTHS

	Duration of Observation		Jan	Feb	Mar	April	May	June.	July	Aug	Sept.	Oct.	Nov.	Dec.
Boston	16 years	Mean	0 96	1 05	1 70	2 97	4 46	5 54	5 98	5 50	4 12	3 16	2 25	1 51
		Maximum	..	..		3 12	5 89	7 01	7 50	7 41	5 13	4.13	3 00	
		Minimum	.	.		2 78	3 35	3 94	4 82	4 25	3 08	2 51	0.66	
Sweetwater	7 years	Mean	2 69	2 41	2 86	4 46	4 96	5 41	6 43	6 74	5 96	4 77	4.51	2 68
		Maximum	3 61	3 53	3 38	5 82	6 14	7 30	8 81	9 02	7 36	6 56	5 53	6 28
		Minimum	1 59	1 35	1 08	3 63	3 45	3 19	3 16	3 07	4 64	3 00	3.35	0 25

Evaporation from water surfaces varies less than does rainfall, the greatest variation from the annual mean at Boston during sixteen years, for instance, being about 13 per cent.

The ratio of the mean monthly to the mean annual evaporation at each place is shown in Table No. 23, as well as the

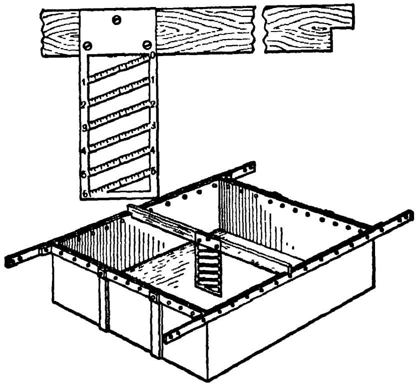


FIG. 19.—Evaporating Pan.

monthly ratios for the year of maximum evaporation at each place.

TABLE No. 23

RATIOS OF MONTHLY TO MEAN ANNUAL EVAPORATION FROM WATER SURFACES AT BOSTON AND SWEETWATER

	Jan	Feb	Mar	April	May	June	July	Aug	Sept	Oct	Nov	Dec
Boston, mean monthly rate	025	027	043	076	114	141	153	141	105	080	057	038
Sweetwater, mean monthly rate	050	045	053	083	092	100	120	125	111	088	083	050
Boston, monthly rate, year of max. evap.	022	024	039	069	086	161	162	170	117	064	051	035
Sweetwater, monthly rate, year of max. evap	043	023	052	098	079	109	148	110	105	110	080	043

These two places represent almost the extremes of the United States, and the monthly ratios for almost any locality will lie between these as limits. It will be noticed that the ratios are much more uniform on the Sweetwater basin than in Boston, owing largely to the more uniform temperature throughout the year; also that in Boston the excessive evaporation during the year of total maximum occurred between June and September, and on the Sweetwater basin between June and October—that is, during the warmest weather.

Evaporation from snow was found at Boston to average .02 inch per day; and that from ice .06 inch per day. On exposed mountain sides and tops it would probably exceed these averages.

Still more important than the evaporation from water is that from soils of different characters; but very few reliable data on this subject have been obtained. The amount of water taken up by vegetation is important, but on this subject also very little definite information is available.

Experiments at Rothamsted, England, showed the annual evaporation from bare soil to be 17.09 inches when the average rainfall was 31.04 inches; and of the 13.95 inches percolating through the soil 9.44 inches was collected between October and March, and 4.51 inches during the seven warmer months. Other English experiments indicated that during October to April the evaporation from grass and earth averaged 1.3 inches a month, and during May to September 3.6 inches a month; being a maximum in May and decreasing more or less uniformly to a minimum in December. The Kansas State Board of Agriculture found that grass and grains consume 3.6 to 4.3 inches per month during the growing season and potatoes about one-fourth as much. Oak trees have been said by certain investigators to consume about 1 inch a month, and fir trees 0.6 inch to 1.2 inches; but concerning the consumption by forests practically nothing definite is known.

In the case of thick coverings of grass or other low vegetation, and forests with their cool shade and leaf carpet, there is probably little direct evaporation from the soil, but most of it is from the vegetation by transpiration. This class of evaporation is greatest during the growing season; and as direct evaporation increases with the temperature, and this is highest during June, July, and August, this season sees the maximum total evaporation. On the other hand, not only is direct evaporation from water and soil and by vegetable transpiration less in the colder months, but the fact that the water is covered with ice and the ground with snow or ice most of the time still further lessens the evaporation. Rafter and others therefore divide the year into three seasons: Storage period, from December to May, inclu-

sive, during which most of the precipitation runs off over the surface, or remains stored in the form of snow. Growing period, from June to August, when most of the precipitation and much of the ground storage is evaporated directly or withdrawn by vegetation. Replenishing period, September to November, during which the ground storage is returned to normal and the direct surface run-off is greater than during the growing period. Such consideration forms the chief reason for beginning a " water year " on December 1st; or, as some do, on September 1st.

Much of the above, with its consideration of snow and cold weather, refers to the northern part of the United States. Data concerning the Southern states are few; but the same general principles would seem to apply. It is possible, however, that for Southern areas conditions would more closely be represented by separating the year into two seasons only, a growing season and a replenishing one.

Sufficient figures have not been and probably cannot be secured to warrant a definite estimate of evaporation directly; but most, even of those given above, were determined by deducting yield from rainfall. Nevertheless it is the evaporation that determines the yield, and not *vice versa,* and if the yield is to be estimated of a catchment area concerning which there are no records of run-off, the method of rainfall minus evaporation seems to be the only one available.

Several formulas have been devised for making such an estimate. Each of these seems to apply to certain catchment areas, all similar in certain essential characteristics; but none is reliable under all conditions. Perhaps as reliable as any is that of Vermeule (consulting engineer of New Jersey State Geological Survey), but this the author has given as applying only to New Jersey, and other sections where temperature, soil and other controlling conditions are similar. His formula is

$$E=(15.5+.16R)(.05T-1.48),$$

in which E is the annual evaporation (including vegetable absorption), R the annual rainfall, and T the mean annual

temperature. The monthly evaporation he represents by the following, in which r is the monthly rainfall:

January.	February.	March.	April.	May.	June.
$.27+.10r$	$.30+.10r$	$.48+\ 10r$	$.87+.10r$	$1.87+.20r$	$2\ 50+\ 25r$

July.	August.	September.	October.	November.	December.
$3.00+.30r$	$2.62+.25r$	$1.63+.20r$	$.88+\ 12r$	$.66+.10r$	$.42+\ 10r$

This formula seems to give very close results for New Jersey, eastern New York, and eastern Pennsylvania streams, but is inapplicable to many sections. For instance, applying this to the basin of the Sweetwater River, Cal., would give an annual evaporation 5 to 15 inches greater than the rainfall. The safest rule, where direct measurements of the stream flow is impossible, is to compare the watershed under consideration with another of known yield and similar in all or most of its characteristics, including precipitation; bearing in mind the effect of variations in elevation, temperature, wind, and other conditions already referred to.

ART. 34. NATURAL STORAGE

There is another factor of yield which affects more the periodic than the total yield, although it has considerable effect upon the latter also. This is the storage in the ground of water, part of which supports vegetation, and part slowly feeds springs and streams; also storage in ponds of surface flow which is gradually, though more quickly, given to the streams. A part of the water so stored in either ground or pond is evaporated; but a considerable proportion of the ground water, and generally of the pond water also, reaches the stream. This storage is not only an important factor in maintaining continuous stream flow and in supporting vegetation, but it has, as stated, some effect upon the total yield, and is valuable also in relieving the storage reservoir of a considerable portion of its duty. Where there is little ground storage, as on dense clay

land, there can be little vegetation, largely because the ground contains no moisture to support it; and since little water soaks into the ground, most of the rainfall runs immediately to the stream. Where a large amount soaks into the ground, considerable of this is taken up by vegetation through its roots and thus abstracted from the yield. We might therefore expect to find a greater total yield from a rocky or clayey soil than from a loamy or sandy one; although it would be more difficult to retain the whole for use, owing to the great quantity flowing off in a very short length of time.

Not only does ground storage develop vegetation, but vegetation, by loosening the soil with its roots and obstructing surface flow, develops ground storage, and hence it follows, both as cause and effect, that the yield from a wooded or cultivated soil is more uniform and less "flashy" than from a bare one.

The water which percolates into a soil descends to a more or less fluctuating ground-water level, which is the surface of the stored water. This surface slopes in the direction in which the ground water is moving, its elevation at any point being governed by that of the outlet and by the amount flowing (the greater the amount the greater being the velocity of flow and hence the slope), the slope increasing with the fineness and density of the soil also. (See also Art. 43.) If a stratum of impervious material lies higher than the elevation at which the ground-water surface would otherwise stand, the ground water will flow above this, generally with greater velocity and consequently furnishing less storage and more varying yield.

Above the ground-water surface proper, some water is held in the soil by capillary attraction for nourishing plant life, and in rather fine-grained soils the amount thus retained may be considerable. As evaporation removes this water from the top soil, capillary attraction draws more from below, to be in turn evaporated; but the amount of water which can thus be raised decreases with the fall of the ground-water surface, and thus, when the interval between rains is long, the upper soil becomes thoroughly dried. The next rain must

then renew this upper supply before contributing any water to the run-off storage. "If the rainfall is sufficient to supply the evaporation and plant growth, the flow from ground water will remain constant, because the head which forces it through the rocks and gravels is constant. When the rain is insufficient, the head will be drawn down and the flow will decrease at a certain fixed rate." Once the draught upon the ground storage is fairly established and the water drawn down, unless the rainfall is greater than it usually is in summer it is all absorbed by the dried earth and does not reach down far enough to increase the head and consequent flow of ground water. "Rainfalls which, if occurring in May, or in the autumn after the ground water has been replenished, would cause violent floods, have no effect at all upon the stream flow when they occur during dry months. This difference in effect cannot be ascribed to direct evaporation, for in the case of concentrated rainfall evaporation has little time to act. It is due to the drawing down of ground water, which leaves a great capacity for absorption of rain by the earth." (Vermeule's Report on "Water Supply, Geological Survey of New Jersey.")

Precipitation as snow, if falling upon unfrozen ground, may gradually turn to water and soak into the soil, or be melted more or less rapidly by sun and rain, when more or less of it will enter the soil. If snow falls on frozen soil, however, it will practically all run off over the surface if melted by a single short rain or warm spell. Thus not only the amount of snowfall, but the previous condition of soil also, affects the amounts of both ground storage and direct run-off.

If a heavy rain storm followed a long wet spell during which the ground had become saturated, practically all of it would run off over the surface. It would thus all be added to the immediate yield (although in the form of a flood which it might be impossible to store), and none to the ground storage. If the balance of the year were dry we might have a year with small rainfall but an abnormally high yield. Or again if the rainfall were light but almost continuous from March to September, keeping the ground surface soaked and vegetation abundant, we might

have, with a high total rainfall, a very high evaporation, giving abnormally low yield. It is apparent, therefore, that the time of year and successive rates and intensities of rainfall have fully as much effect upon yield as the total precipitation for the year.

The absorption capacity varies with different soils, and also the amount of water yielded. A coarse gravel will yield almost its entire contents, while fine sand or clay will yield practically none, retaining it all by capillary attraction. Table No. 24, from Schubler, gives results obtained from various soils. From this table it would appear that, given the same conditions as to vegetation, exposure, climate, etc., the evaporation from all soils would be quite similar.

Table No. 24

CAPACITY FOR ABSORPTION AND YIELDING UP OF WATER POSSESSED BY VARIOUS SOILS. (FROM SCHUBLER)

Soil.	Water Absorbed by 100 Parts of Soil after Drying at 40° or 50° Fahr.	Percentage of Water Evaporated in 4 Hours at 56.7° Fahr.	Parts of Water in 100 Parts of Soil Evaporated in 4 Hours (Soil Saturated).
Siliceous sand	25	88	22
Gypseous soil	27		
Calcareous sand	29	75 9	22
Barren clay	40	52	21
Fertile clay	50	45 7	23
Loamy clay (or clayey soil)	60	34.9	21
Pure clay	70		
Fine calcareous soil	85	28.6	24
Humus	190	20.2	38
Magnesian soil	156		
Garden soil	89		

The least flow of the Connecticut river is equivalent to about .05 cubic foot per second per square mile of watershed, which is maintained during probably a month at least of no rainfall and three months when the rainfall no more than equals the evaporation and requirements of vegetation. The flow must, during this time, be maintained by the ground-water

storage, and is equivalent to about 1.7 inches of water over the entire area. If the soil furnishing ground storage averages an absorption capacity of 33 per cent, the ground-storage surface would be lowered about 5 inches on an average. But not all the ground furnishes storage, and probably not more than 50 per cent of that held is yielded to the stream; also the lowering increases with the distance from the river; so that in some places it may amount to 3 feet or more.

In a fairly wet or rainy season the ground-water surface may be raised 5 or 10 feet, even reaching the surface in many places and producing ponds or swamps.

Most storage reservoirs have not water-tight shores, and as the water rises in the reservoir the ground-water level in the vicinity rises also, and ground storage is obtained in addition to that in the reservoir. When the reservoir water is drawn down this ground storage also is drawn upon, and more water is yielded than the capacity of the reservoir.

Art. 35. Yield and Run-off

The *yield* of a given catchment area is that part of the total rainfall thereon that is not used by vegetation or evaporated, either directly or by vegetable transpiration. The term *run-off* should probably, strictly speaking, be confined to that which runs off over the surface; but this is so inseparable from the ground flow near the surface (the same particles of water often flowing first on the surface, later through the ground, and again on the surface), that it is generally used to refer to all the water that reaches the stream or reservoir to which the area drains. In many cases a part—generally a small percentage—of the ground storage enters strata of sandstone or other rock, or of sand or gravel, that lie deeper than the lowest point of the catchment area, and such water may flow through such strata and reach the surface at some distant point, some strata conducting the water for hundreds of miles before it finds an outlet. Such strata are the sources from which water is obtained by deep wells. Water enters them slowly at points where they outcrop on the

surface, most of it probably from streams that flow over them, or from deposits of porous material that overlie them and hold water continuously in contact with them. Such water is part of the yield but not of the run-off.

Rainfall minus evaporation equals yield, but only when these are totaled for long periods. The equation is seldom true for monthly or even for yearly records, since it fails to consider the varying amount of water held in storage in the ground. The average run-off for a series of years is of only minor importance unless storage is available for holding the entire surplus of the longest series of wet years and supplying it during the succeeding dry ones—which is seldom if ever possible. What is more essential is ability to forecast what will be the minimum run-off for a month, a year, or a series of years; also what will be the maximum run-off in any minute, five-minute, or sixty-minute period. The latter is essential in designing spillways, flumes, etc., the former in determining the amount of water available unfailingly for use and the amount of storage necessary or practicable.

Knowledge of the conditions that affect run-off is important in order that an approximate estimate of it may be made directly from consideration of such conditions, and also that an area of known run-off may be selected for comparison where the conditions most influencing run-off are practically the same as those of the area under investigation. Both methods should be employed, the latter being given more weight if the similarity of such conditions warrant. The affecting conditions may be briefly summarized as follows:

The total annual precipitation, how much is rain and how much snow, its distribution throughout the year, and whether it comes as frequent light rainfalls or as occasional heavy storms.*

The temperature by months and seasons, whether or not high temperatures occur during times of greatest precipitation (thus increasing evaporation), whether the winter temperatures favor

* The U. S. Geological Survey divides its run-off records into climatic years that begin with October 1.

the long retention or rapid melting of snow and ice, and whether the ground is apt to be frozen at the time of the spring rains.

Whether the area lies in the path of storms, and whether its greater axis lies parallel or at right angles to such path. (These affect the amount of precipitation.)

The rate of inclination of the drainage area surface, and whether it is smooth or rugged, whether the area is large or small, long and narrow or short and broad. (These affect the rate of concentration of the run-off into the stream or reservoirs.)

Whether the surface is bare or covered with vegetation, and the nature of such vegetation, and whether the soil is kept porous by cultivation. (These affect chiefly the evaporation, both direct and by transpiration.)

Direction and intensity of prevailing winds. (These affect evaporation.)

Geology of the area, whether the soil be porous or impervious, whether the porous deposits be shallow or deep, level or inclined, and whether the outlets of the water they receive are in the area in question, are below a proposed dam site in that valley, or are outside of it altogether. (These affect ground storage and the amount thereof that is recoverable.)

The existence of lakes, swamps or other surface storage agencies. (These produce more uniform run-off, also increase evaporation.)

It is evident that, with all these variables to modify it, catchment areas will show wide divergences in their amounts of run-off, either total or per acre. A common method is to estimate the annual run-off as a percentage of the annual rainfall. In a given section of the country and where topographical conditions are similar, there will often be a fair uniformity in this percentage; for instance, in the eastern parts of Massachusetts, New York, and New Jersey, the averages range from about 45 to 55 per cent; but even here some areas will fall to 40 and others rise to 60 or more. In the Southwest the percentage may be as low as 10 or 15 per cent. Table No. 25 gives the average percentages for several streams; it also shows the small percentages of run-off during the growing season, and that

Table No. 25

RUN-OFF FROM A NUMBER OF WATERSHEDS

Watershed.	Area in Sq. Mi.	Average Percentage of Rainfall Reaching Streams.													Remarks.
		Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Year.	
Perkiomen creek, Pa....	152	79	85	108	81	40	25	18	33	28	25	46	61	51.6	For 10 yrs. 71% cultivated.
Neshaminy creek, Pa....	139.3	90	96	106	72	31	15	13	20	18	19	40	76	48.7	For 10 yrs. 92% cultivated.
Tohickon creek, Pa.....	102.2	105	115	124	82	34	21	18	25	29	24	53	72	59.6	For 10 yrs. 72% cultivated.
Connecticut river.......	11,083	59.1	65.8	76.3	145.0	132.2	36.5	21.3	21.8	29.3	28.3	44.8	60.7	56.5	For 15 years.
Sudbury river, Mass....	75.199	49.1	78.2	109.6	109.1	62.3	29.1	8.9	13.0	14.2	23.1	39.5	52.5	49.5	For 16 yrs. Hilly, steep, some swamps.
Cochituate river, Mass..	18.87	53.1	71.9	84.6	83.8	47.9	27.9	13.1	17.7	23.5	23.6	34.0	49.9	43.8	For 29 years. Flat, sandy.
Potomac river (Maryland and W. Virginia).	11,043	65.2	98.5	84.9	104.1	44.7	38.2	23.1	21.3	25.0	40.5	54.3	72.3	53.0	For 6 years. Steep valleys; few lakes.
Savannah river.........	7,294	57.6	74.6	64.4	95.7	37.3	31.8	22.8	42.7	46.8	41.6	79.1	48.1	48.9	
Arkansas river.........	3,060	...	...	...	...	...	..	..	..	..	..	..	..	30.9	
Salt river.............	12,260	...	...	...	...	...	..	..	..	..	..	..	..	29.2	
Upper Snake river.....	10,100	...	...	...	...	...	..	..	..	..	..	..	..	67.6	
Bear river.............	6,000	...	...	...	...	...	..	..	..	..	..	..	..	41.5	
Owyhee river..........	9,875	...	...	...	..	...	..	..	..	..	..	..	..	16.2	

during some spring months the run-off is regularly greater than the rainfall. On the Sudbury the annual yield has varied from 31.9 to 62.2 per cent of the precipitation. " The percentages depend upon the distribution of rainfall throughout the year. A heavy summer rainfall and a light winter rainfall mean a small percentage of collection; and, conversely, a light summer and a

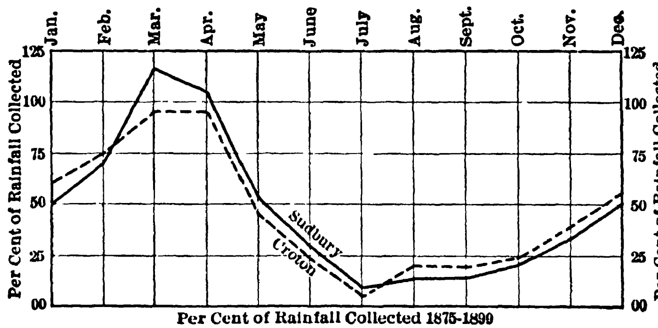


FIG 20 —Per Cent of Rainfall Collected in 1875–1899 on the Sudbury and Croton Watersheds

By X. H Goodnough, Journal N E W.W Ass'n, 1915.

heavy winter rainfall mean a large percentage of collection; so that the total rainfall for the year is but a partial index to the yield of a watershed." "It may be said that, for systems which

FIG. 21*a*.—Precipitation and Percentage Collected; Sudbury Watershed.

depend upon storage, it is not the summer droughts which are to be dreaded, but the winter and spring droughts; for it is the

flow in these months upon which we depend to fill the reservoirs." (Desmond FitzGerald, in Transactions of American Society of Civil Engineers, Vol. XXVII.)

In California the percentage of rainfall collected annually may vary from nothing when the rainfall is less than 20 inches,

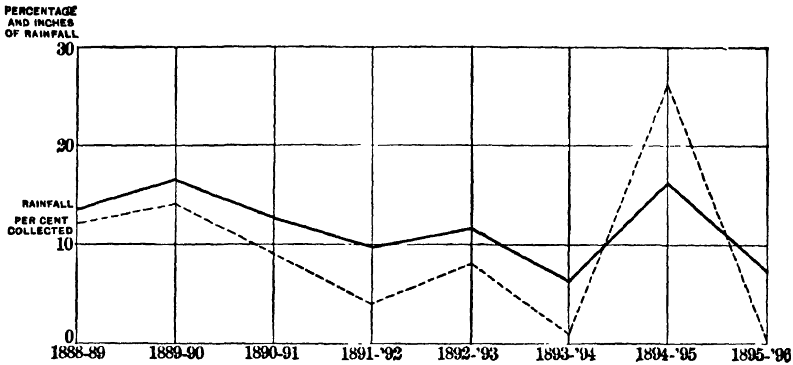


FIG. 21*b*.—Precipitation and Percentage Collected; Sweetwater Basin.

to 60 per cent in years of heavy rainfall, averaging not far from 30 per cent. In Southern California the average for seven years was 10.6 per cent.

The following table shows the maximum and minimum yield for two watersheds:

TABLE NO. 26

MAXIMUM, MINIMUM, AND MEAN YIELD; CONNECTICUT AND POTOMAC WATERSHEDS

	Jan.	Feb.	Mar	April	May	June	July	Aug	Sept	Oct.	Nov	Dec
CONNECTICUT RIVER, 13 YRS												
Rainfall, inches depth... ...	3 27	3.10	3 94	3 26	3 17	4 00	4 79	4 87	3 04	3 93	3 93	3 3'
Max yield, inches depth...	5 70	4 06	5 64	7 61	6 54	3 11	2 32	2 52	2 24	2.81	3 60	4 9
Min yield, inches depth ...	0 72	0 77	0 90	2 61	1 90	0 79	0 69	0 70	0 66	0 71	0 67	0 7
Mean yield, inches depth ...	1 93	2 04	3 00	4 73	4 19	1 46	1 02	1 06	0 89	1 11	1 76	2 06
Mean percentage yielded. .	59 1	65 8	76 3	145 0	132 2	36 5	21 3	21 8	29 3	28 3	44 8	60 7
POTOMAC RIVER, 6 YEARS.												
Rainfall, inches depth....	3 21	3 35	4 39	3 48	5 11	5 25	4 89	3 81	3 86	2 65	2 88	2 59
Max yield, sec ft per sq mi	14 00	15 81	17 10	20 87	18 60	42 88	19 36	5 94	63	14 50	15 25	9 48
Min. yield, sec ft. per sq mi	0 24	0 22	0 25	0 25	0 23	0 25	0 25	0 27	27	0 27	0 27	0 27
Mean yield, sec ft per sq mi	1.81	3 22	3 14	3 15	2 04	1.75	0 87	0 67	0 95	1 07	1 61	1 15
Mean percentage yielded	65 2	98.5	84 9	104 1	44 7	38 2	23 1	21 3	25 0	40 5	54 3	72 3

From examples of such wide variation it is evident that the use of a mean percentage of the rainfall for estimating the run-off will give only very approximate results; particularly since the lowest percentages usually occur during the years of minimum precipitation. (See Figs. 20 and 21.)

Run-off is sometimes expressed in cubic feet per second per square mile, as in Tables Nos. 26, 27, and 28. Since run-off is a product of the rainfall and the percentage of this reaching the streams, both of which are variables, the records of stream flow might be expected to show greater variation than either of these, which is seen to be the case.

Table No. 27

RUN-OFF FROM SEVERAL DRAINAGE BASINS

Watershed.	Length of Observation, Years.	Area, Sq Mi.	Run-off, Cu. Ft per Sec. per Sq. Mi		
			Max.	Min.	Mean.
Subdury.	16	75,199	44 26	0 04	1 669
Connecticut.	15	10,234	20 00	.	1 86
Croton.	14	375	70 83	0 14	1 55
Perkiomen.	7	152	69 20	0 05	1 90
Raritan.	1	879	27 00	0 14	1 72
Passaic.	17	773	24 80	0 17	2 58
Potomac	6	11,043	42 60	0 17	1 85
Savannah.	8	7 294	41 2	0 27	1 64
Ohio.	1	200,500	6 17	0 27	2 10
Missouri	12	526,500	8 52	0 60	2 16
Arkansas	5	3,060	1 55	0 06	0 27
Rio Grande.	3	30,000	0 55	0 00	0 06
Gila.	1	13,750	0 46	0 00	0.04
West Carson.	2	70	18 34	0 50	2 39
Sevier (Utah).	3	5 595	0 42	0 01	0 09

That the run-off per square mile from two watersheds should be the same, it would be necessary that they be similar in all the conditions given above as affecting this. Two adjacent watersheds may differ greatly in some of these conditions, and hence in yield. For instance, we find the proportion of rainfall yielded by three Pennsylvania watersheds within a few miles of each other to vary as 51.6, 48.7, and 59.6 (Table No. 25), or a

difference of 22 per cent between extremes. Two adjacent Massachusetts watersheds varied in percentage of yield by 12 per cent, or as 49.5 and 43.8. In central California the mean yield is not far from 30 per cent of the precipitation, while in the southern part of the same state the mean for seven years was but 10.6 per cent.

TABLE NO. 28

RATIO OF MONTHLY TO MEAN RUN-OFF

	Jan	Feb	Mar	April	May.	June	July	Aug	Sept	Oct	Nov	Dec	Mean Monthly Run-off
Mean of 3 New England watersheds for 13 yrs cu ft per sec per sq mile	1 866	3 042	3 555	2 533	1 518	0 772	0 374	0 569	0 528	0 838	1 278	1 703	1 539
Ratio of monthly to mean	1 20	1 97	2 31	1 65	0 98	0 50	0 24	0 36	0 34	0 54	0 83	1 10	
Boston, for 20 yrs ratio of monthly to mean	1 15	1 80	2 68	1 91	1 10	0 46	0 17	0 28	0 23	0 47	0 81	0 94	1 634

Although these data show a great variation in percentage of rainfall yielded in different parts of the country, and in different years on the same watershed, the average yield in a given section of country is fairly uniform. For instance, in the New England and Central Atlantic states the yield generally averages from 44 to 60 per cent of the precipitation (this paragraph refers to yearly totals only), and the extreme variation in any one place is about 25 per cent of its mean, so that the use of the average rainfall and average percentage yielded would very probably give a result in error not more than 30 or 40 per cent. Again, if we compare the second-feet per square mile, we find in the same locality a mean yield of about 1.6 second-feet with a maximum variation from this of about 40 to 50 per cent. By using judgment in selecting for comparison a watershed with characteristics similar to the one in question, the probable error may be reduced in many cases to 10 or 15 per cent. Below this it is hardly practicable to go, except by obtaining long-period records of yield of the area in question.

Where no watershed with similar conditions can be found, the decision must be a more or less arbitrary one, based upon information obtained from the oldest inhabitants, observed stream-channels and ponds, the location of the place with reference to the coast, sea level, mountain ranges, etc., the character of soil and topography, and the vegetation.

The measured yield of the area in question for a long series of years forms of course the most reliable basis of estimate; the next is the known yield of similar areas in the same district; and from precipitation, either gaged at the locality in question or estimated from the district rate, the yield may be estimated either as a certain percentage of this, or by deducting the probable evaporation according to Vermeule's rule for the Eastern states, or a similar one formulated for the locality in question. In fact, it is desirable to estimate by each of these methods and compare results, giving them relative weight in the above general order.

With reference to yield throughout the United States the general statement is made in the U. S. Weather Bureau Reports that " for the area of the United States east of the 95th meridian (Omaha and Galveston) the run-off is from 35 to 50 per cent of the total rainfall. It appears to be largest in the vicinity of the Great Lakes, and to diminish from this region slowly to the south and east, and rapidly towards the west. In the lower peninsula of Michigan, for instance, the run-off is 50 per cent of the total rainfall. Along the Gulf coast it appears to be only from 30 to 40 per cent, and along the Atlantic coast it probably varies from 30 to 50 per cent. In general, for the interior states east of the 95th meridian the run-off is between 40 and 50 per cent of the total rainfall.

" As soon as we cross the 95th meridian westward we find a very sharp fall in the percentage of run-off to the total rainfall. For the band extending north and south between the 95th and 105th meridians this percentage varies from 10 to 25 per cent, and over Iowa is about 33 per cent. The percentage is highest at the northern end of the band indicated, and lowest at the southern end. Going still farther westward we come

to another very marked area, that of the Continental Divide; here the percentage of run-off suddenly increases, reaching the highest figure to be found in the United States. From Montana to Colorado it varies from 60 to 70 per cent of the total rainfall. In New Mexico it falls to about 33 per cent. This is evidently on account of the easy flow of water from the mountain ranges in the area in question. West of the Divide the run-off is again small, being only 15 or 20 per cent in Arizona and Nevada, about 30 per cent in Idaho, and nearly 50 per cent in Utah. Utah, it seems from its topography, partakes of the character of the band lying just to the east of it. Along the Pacific coast the run-off is about 25 per cent in Oregon, 30 per cent in Washington, and between 45 and 50 per cent in California" (the nothern part only).

The above is a very general statement, and, particularly in the far West, is subject to many variations and exceptions, some of which are recorded in the preceding tables.

(It may be convenient to remember as an approximation that the average yield of New England watersheds in general ranges between 300,000 and 600,000 gallons per day per square mile.)

The apparent non-uniformity of the data relating to yield has been emphasized, not because such data or the methods of using them should not be made a basis for estimating, but to call attention to the fact that the estimates cannot be expected to give more than approximate results. When the data are meager or unreliable, any works based upon them should be more or less tentative in design and construction, and capable of being modified as the developing conditions require.

The data required for estimating yield will be found in the annual Reports on Hydrography of the U. S. Geological Survey, the Meteorological Reports of the Weather Bureau, reports of the Geological Surveys of several states, of the Departments of Agriculture of several Western states, and the reports of the water-works departments of various cities.

Art. 36. Run-off from Storms

If a storage reservoir is to retain all the yield of a given area it must be sufficiently large to hold the entire run-off from the heaviest downpours, over and above what may be already stored in it during the wettest years. Probably no reservoir has been constructed to do this, and the overflow must therefore be capable of passing the maximum storm run-off. It is hence necessary to know what the maximum rate of run-off from the heaviest storms will be. The maximum run-off from large drainage areas in the North frequently occurs, not from the maximum precipitation, but from a heavy warm rain falling upon a ground covered with considerable snow—the spring freshets. At this time about 4 to 8 inches of snow is equivalent to 1 inch of water. Hence 2 feet of snow carried off in twenty-four hours by a rain would add to the run-off the equivalent of 3 to 6 inches of additional rainfall running off in that time, or of $\frac{1}{8}$ to $\frac{1}{4}$ inch per hour additional to the actual precipitation, if all of the latter be considered to run off. Probably the addition for snow of $\frac{1}{8}$ to $\frac{1}{4}$ inch per hour to the run-off from the rainfall will be sufficient for most cases. If the surface under the snow be frozen, 90 per cent or more of the rainfall may be yielded in addition to this.

The maximum rates of rainfall generally last for five to fifteen minutes only. Assuming a velocity of flow over the surface of 2 feet per second and in the stream of 4 feet (although this may reach 8 feet per second), with 1000 feet of watershed above the head of the creek, at the end of fifteen minutes of maximum rainfall the first rain falling at the head of the basin would have traveled 2600 feet, and at all points above its position at that time the run-off would be that due to the maximum rate of rainfall. But at all points more than 2600 feet from the basin head, the run-off from only the nearer part of the basin would be at that rate, the upper part contributing to the flow at a rate due to the rainfall previous to the beginning of the maximum. Thus, at 3800 feet from the basin head the flow would be that due to a rainfall for a certain five minutes

on the upper 600 feet of basin, for the next five minutes on the next lower 800 feet of basin, for the next five minutes on the next lower 1200 feet, and for the last five minutes on the nearest 1200 feet of basin. At any point in the run-off channel the maximum flow will be approximately the product of area of shed above such point, the maximum average rate of precipitation for the time consumed by the run-off in flowing to this point from the most distant point of the basin, and the proportion of rainfall running off. The determination requires a knowledge or assumption of the proportion of rain yielded as surface flow, and the velocity of flow of the maximum run-off. No close approximation to this latter can be made except from actual observation on each watershed. The run-off from heavy downpours of short duration may be 60 to 70 per cent on steep clay or stony hillsides, and even 90 per cent or more on rocky or frozen ground; while for flat slopes of loose soil 30 per cent may be the maximum amount.

It is evident that the larger the drainage area the less will be the rate of precipitation used and hence the rate of run-off per square mile. Distance is as large a factor of this rate as is area. From a small drainage basin 10,000 feet long, with a maximum velocity of run-off of 2 feet per second, the length of time for which the maximum rate of rainfall is to be considered is 5000 seconds, or 1 hour $23\frac{1}{3}$ minutes. At Mt. Carmel on July 2, 1897, 5.03 inches fell in $1\frac{1}{2}$ hours. Assuming this as a maximum rate and 50 per cent running off, we have a run-off of 1.68 cubic feet per second per acre, or 1075 per square mile. This rate of precipitation—3.36 inches per hour—can probably be considered a maximum for the New England and North Atlantic states.

Several empirical formulas have been devised to express the maximum rate of run-off from a given area. A few of these are:

Fanning's formula.................. $Q=200M^{\frac{5}{6}}$;

Dredge's formula.................. $Q=1300\dfrac{M}{L^{\frac{2}{3}}}$;

Col. Dickens' formula.............. $Q=CM^{\frac{3}{4}}$;

in which Q = cubic feet per second yielded from the whole area;
M = area of watershed in square miles;
L = length of watershed in miles;
C = 200 in flat country, 250 in mixed country, 300 in hilly country, for a rainfall of 3.5 to 4 inches; or 300 to 350 for a 6-inch rainfall.

In the example on the previous page, if the watershed contain 3 square miles, being 1.9 miles long, the maximum rate of run-off as calculated by the above formulas would be:

By Fanning,	$Q = 500$ cubic feet per second;
Dredge,	$Q = 2550$
Dickens,	$Q = 684$
above solution,	$Q = 3225$

As the size of the drainage area increases and the rate of precipitation used consequently decreases, these formulas will give quantities more nearly approaching those obtained by the above analytical method, and when an area 20 or 25 miles in length is involved, that of Fanning and the analytical method would give approximately similar results. But for small areas the above formulas will generally give unsafe results, and the method outlined is recommended for designing waste-weirs. (Several serious wash-outs and destructions of dams have resulted from the designing of waste-weirs or spillways by use of the above formulas.)

ART. 37. STORAGE

In Art. 29 was given an illustration of the reason for storage and the amount required for private use. The reason is the same for storing public supplies, but the amount stored is of course vastly larger, and the length of drought provided for is usually longer. The storage reservoirs for the San Francisco water supply have a united capacity equal to the total consumption for three years. In the East two-thirds this capacity or less would be considered sufficient, however, the annual precipitation being more uniform.

A measurement of the drainage area having been obtained,

and a decision formed as to the probable average, minimum and maximum yield, by both year and cycle, and the consumption to be provided for being determined, a calculation of the storage required can be made. If the minimum annual yield is equal to or greater than the desired consumption, storage for only the dry season of one year of drought is required; if the minimum *daily* yield equals the maximum daily consumption, no storage is required; but if the assumed consumption is nearly or quite equal to the mean yield, all the surplus from the years of greatest rainfall must be stored and carried until times of drought. In many cases it may be advisable to construct a reservoir in such a location or of such capacity that it is capable of tiding over one dry season only, if this be ample for the consumption for a few years to come; and when the capacity of the reservoir is almost reached by the consumption, a reservoir of larger capacity may be built, and better adapted to the watershed in question because based on data meantime collected, the interest on the additional sum which a larger original reservoir would have cost being saved during this period.

In making the calculation for storage, evaporation from the reservoir must be considered, and may be added to the consumption. It is of course proportional to the area of water surface in the reservoir, and for the preliminary calculation this area must be assumed. For average conditions in the New England and Middle Atlantic states the maximum reservoir required would have a capacity 150 to 175 per cent of the mean annual yield. An inspection of the yield for a series of years at any location will give the approximate capacity required to permit the consumption to equal the average yield. This capacity divided by the average depth of reservoir will give its area. About one-tenth that of the drainage area may be considered a maximum which will rarely be exceeded; and one-twentieth may be taken as an ordinary maximum. The amount of evaporation in cubic feet will be this area times the rate of evaporation, both expressed in feet. The annual rate of evaporation east of the Mississippi probably never exceeds the mean rate by more than 10 or 15 per cent, although in the Western states it may be

30 or 35 per cent. (See Table No. 21.) To be on the safe side, the maximum annual rate may be used, and apportioned to the months if monthly yield and consumption are to be used in the estimate.

On the Sudbury basin the mean evaporation from water in inches of depth was 171 per cent of the mean yield in inches from the entire catchment area. Assuming the reservoir area as $\frac{1}{20}$ the watershed, we would have the loss from it by evaporation about 8.5 per cent of the yield; and since the size of the reservoir will vary with the consumption, which cannot exceed the mean yield, we may assume with little error that the evaporation loss from similarly located reservoirs will not exceed 8 to 10 per cent of the consumption.

There will be some loss from a reservoir due to seepage through the dam. Seepage into the ground above the dam may be considered as additional storage; for, although a part of this may be absorbed by vegetation, the proportion will probably be little if any greater than would have been the loss by evaporation had it remained in the reservoir. Through a masonry dam there will be a little loss, but it should be inappreciable. Through an earthen embankment, however, the loss may be considerable. The amount so lost will depend upon the character of this embankment, which should be so constructed that the daily seepage shall not be more than 10 gallons per square foot of vertical longitudinal section of embankment. If the reservoir be ten times as long as the length of the dam, and this length be 100 times the average height of the dam, this would give a daily loss by seepage of .01 gallon per square foot of reservoir area, or 3.65 gallons per square foot yearly, or say 6 inches; or about about 1.3 per cent of the yield. (These figures are for New England only.) With good materials and workmanship the seepage may be reduced to 5 or even 3 gallons per vertical square foot of embankment.

Evaporation and seepage combined may be assumed as not exceeding 10 per cent of the consumption for the New England and Middle Atlantic states, which will be a safe figure for use when accurate data are not obtainable. The run-off should

be decreased by this percentage, or one similarly obtained, in estimating storage and *available* run-off. For example, to estimate the maximum consumption available from the Sudbury catchment area: we have a mean yield of 30,003,580,000 gallons per annum or 82,000,200 gallons per day. Allowing 10 per cent for loss as above, we have 27,276,000,000 gallons per annum or 74,727,000 gallons per day as the maximum available supply.

A convenient method of calculating the storage required on a given catchment area of known or assumed yield, to meet different rates of consumption, is the graphical or "mass diagram" one, the cumulative yield from the beginning of a dry period to the end of each month in succession being plotted on the ordinate of that month. Such a method is shown in Fig. 22, using a cycle of the dryest eleven years on the Sudbury watershed, viz., 1878–88. (See Fig. 21*a*.) The wavy line is the curve of cumulative yield, plotted from the records. (From this curve the greater yield in each winter and spring is very apparent.) A straight line *FF'* is drawn at an angle representing 30,000,000,000 gallons per year (the mean yield, also the assumed consumption plus loss by evaporation and seepage), and so located as to be tangent to the curve and nowhere intersecting it. The vertical distance between *FF'* and the curve at any point represents the amount which must be in the storage reservoir at that time if the assumed amount of consumption is to be continuously furnished. Thus it appears that at the beginning of this eleven-year cycle the reservoir must contain about 37,000,000,000 gallons or more than one year's consumption; and that the capacity of the reservoir must be at least 56,500,000,000 gallons, being full during the spring of 1879. After this the reservoir becomes less and less full until November, 1885, when it becomes empty, but after which time the precipitation is sufficient for the consumption. An inspection of the sixteen-year curve, Fig. 21*a*, shows that the supply required in the reservoir on January 1, 1878, would not have been provided by the yield of the three previous years, but might have been accumulating for several years back.

If we assume the reservoir empty on January 1, 1878, we draw a straight line from C tangent to the curve and cutting it nowhere. This line is shown by CAC', representing by its angle of slope 24,500,000,000 gallons annually, or about 67,000,000 gallons per day; less 10 per cent loss by evaporation and seepage gives 61,000,000 gallons per day available. We

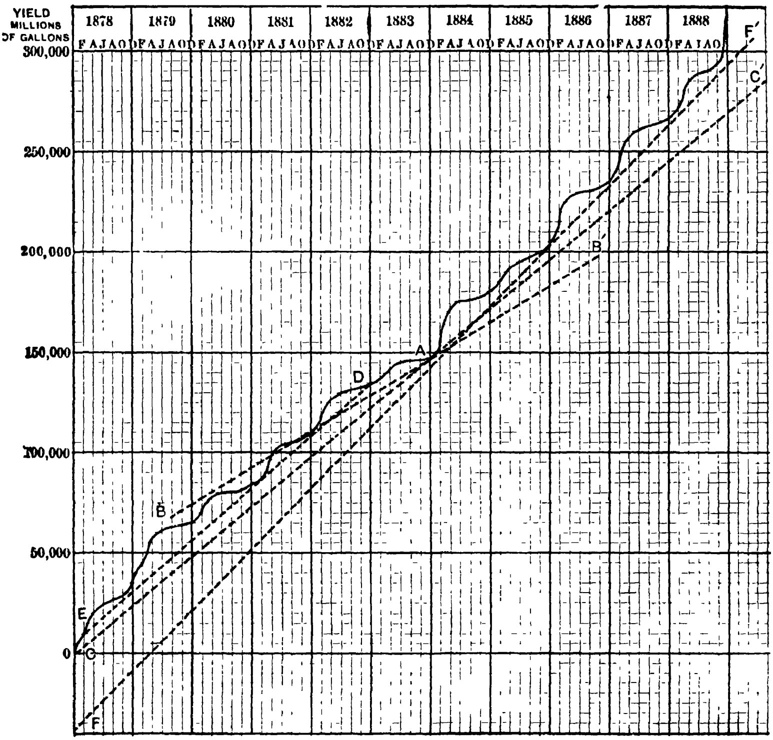


FIG. 22.—Yield and Storage Diagram; Sudbury Watershed.

see by AC' that this rate is more than provided by the yield after January 1, 1884. We also see that the maximum storage required is 27,000,000,000 gallons, and the reservoir would again be full and wasting water in April of 1887 and all through the spring of 1888. It is of course not advisable ever to permit the reservoir to become entirely empty, and a storage of at least 30,000,000,000 gallons should be provided in this case, and double this amount in the former one.

If but 10,000,000,000 gallons storage is provided, what will be the maximum uniform rate of consumption made possible? Place one end of a thread at the deepest loop in the curve, *A*, and swing it towards the left until the greatest vertical distance between the thread and any point of the curve above it is 10,000,000,000 gallons—as the line *AB*. As a check, continue this line towards the right end of the curve making *BAB′* a straight line. No part of *AB′* should come above the right half of the curve; and if it does, either *A* is not at the period of greatest drought, or the storage is unnecessarily large. The rate represented by *AB* is about 17,600,000,000 gallons per year, or a consumption of 43,800,000 gallons per day.

This curve may also be used for finding the total, and mean rate of, yield for any length of time. Thus, from March, 1878 (*E*), to December, 1882 (*D*), inclusive, the total yield was 138,700 −14,700, or 124,000 million gallons; and the mean rate, represented by the angle of the line *DE*, was 25,700 million gallons per annum.

This method is sufficiently accurate for all practical purposes, except that allowance is not made for the variations in monthly consumption and evaporation. An approximate determination may, however, be made by diagram, and more accurate figures obtained by calculation, as follows:

Taking the case of a reservoir with 10,000,000,000 gallons capacity (see table on opposite page), we see that the first storage begins in the middle of January, 1882, and we may begin our table there. Quantities are in millions of gallons.

The monthly yield in this was taken from the record (Trans. Am. Soc. C. E., Vol. XXVII. page 276); the consumption is found by dividing 17,600 million gallons by 1.10 (10 per cent for loss from reservoir), and the quotient by 12 for the monthly mean consumption; the rate for each month being the product of this by the percentage given in Table No. 5. The loss by percolation is taken as 1½ per cent of the mean consumption. And the evaporation is found by multiplying 8½ per cent of the mean consumption by the factors in Table No. 23, the sum of these losses and the consumption being given in the third column.

1 Month.	2 Amount of Yield, Mil. Gals.	3 Consumption and Loss.	4 Surplus Added to Reservoir	5 Deficiency Supplied from Reservoir.	6 Amount in Reservoir at End of Month.
January, 1882 (15 days).	1446	607.3	838.7	....	838.7
February............	5060	1257.7	3802.3	....	4641.0
March....	6618	1239.3	5378.7	....	10019.7
April.	1957	1324.9	632.1	.....	10000
May.................	3011	1483.6	1527.4	......	10000
June.................	1193	1748.0	. .	555.0	9445.0
July...............	201	1844.2	.....	1643.2	7801.8
August..... ..	129	1748.0	......	1619.0	6182.8
September..... .	691	1631.9	......	940.9	5241.9
October.	697.7	1477.3	.....	779.6	4462.3
November...... .	472.3	1350.3	....	878.0	3584.3
December.	733.6	1299.5	......	565.9	3018.4
January, 1883...... .	780.5	1214.7	.....	434.2	2584.2
February....	2174.4	1257.7	916.7	......	3500.9
March...............	3755.0	1239.3	2515.7	....	6016.6
April................	3044.5	1324.9	1719.6	.....	7736.2
May.................	2185.3	1483.6	701.7	....	8437.9
June.................	676.9	1748.0	.....	1071.1	7366.8
July	268.9	1844.2	......	1575.3	5791.5
August...	183.1	1748.0	.....	1564.9	4226.6
September..........	205.9	1631.9	......	1426.0	2800.6
October.	433.2	1477.3	. .	1044.1	1756.5
November...........	461.7	1350.3	......	888.6	867.9
December....	451.1	1299.5	.....	848.4	19.5
January, 1884...... .	2319.9	1214.7	1105.2	....	1124.7
February............	6197.3	1257.7	4939.6	......	6064.3
March	8824.3	1239.3	7585.0	....	10000
April.	6437.3	1324.9	5112.4	....	10000

It is seen that by December 31, 1883, the reserve is reduced to 19.5 million gallons, but is quickly brought up to the limit of the reservoir by the spring run-off.

Another method of making this calculation is to correct the monthly yield of the catchment area for the greater or less yield of the ponds or other water-surfaces thereon, including the reservoir, and to consider only the consumption as being deducted from the reservoir. For this purpose the area of water surfaces relative to that of the entire catchment area must be known, and also the yield of land surfaces. To the land-surface yield over the entire area (assuming seepage from ponds to contribute as much as percolation from rainfall on an equal

area) is added the net yield or loss of each month from the water areas.

The ponds and other bodies of water on the drainage area evaporate more water than do the earth. In New England the monthly rainfall minus monthly evaporation on a water surface averages as follows, in inches of depth:

Jan.	Feb	Mar	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Year.
2 79	2.87	2 50	0 31	−1 09	−2 29	−1 91	−1 10	−0 79	1 09	1 67	1 91	5 96

It appears from this that during May to September the evaporation from a pond is greater than the rainfall upon it; and that during an average year the excess of rain, or yield, is about 6 inches only. In some sections of the country the evaporation is ten times the rainfall; and at Yuma, Ariz., it is thirty times; but these sections are limited in area.

Art. 38. Quality of Surface Waters

The most dangerous impurities to be found in surface-water are those due to pathogenic bacteria, which are ordinarily, if not invariably, derived from human excreta; and a watershed should be carefully examined for and guarded against such contamination. No surface privies or overflowing cesspools should be permitted. Deep, tight cesspools at a distance from any stream, and from any well (since the health of occupants of the watershed is very important to the consumers), should be compulsory for any scattered occupants; and the use of night-soil as fertilizers should not be permitted within such drainage area. If there is a village or any considerable congregation of houses on the watershed, these should be provided with sewers, and the sewage either so treated that no germs can reach the reservoir, or else discharged beyond the drainage area or below all impounding reservoirs. This is very important, since many an epidemic of typhoid fever has been traced to a single case upon a watershed. Epidemics of considerable violence have been due to the depositing upon the snow of the excreta of a typhoid

patient, which were washed into the reservoir with the spring rains.

It is generally in the spring that the greatest amount of impurity is found in surface water, when that absorbed by snow from the air and ground (which absorption is continued while the snow lies) is added to that of the rainfall, and all passes over the frozen ground without any of the purification effected by underground flow.

Surface water, when stored in reservoirs, is subject to certain changes, most of them advantageous but some otherwise. Much of the matter in suspension is here deposited if the reservoir be of such size that there are no considerable currents. Together with the coarser matter, many bacteria may be carried down, probably not on account of their weight but because their food-matter is settling to the bottom. At Oberlin, O., for example, the number of bacteria was found to be reduced from 2000 per cubic centimeter in a 15,000,000-gallon reservoir to 426 in the effluent. In the Chestnut Hill reservoir (Boston) the average number of bacteria found at the surface, middle, and bottom were 77, 246, and 319 respectively; the surface water at no time containing more than one half the number found at the bottom. The benefit of sedimentation to a water supply is illustrated by the typhoid-fever epidemics at Philadelphia in 1891 to 1899, where the highest mortality was almost invariably found where water was pumped directly to the consumers, and the lowest where the capacity of the reservoir relative to the consumption was greatest.

Waters entering a reservoir from soils of different character, and as both surface and ground flow, will possess different characteristics. These waters largely intermingle, the more polluted being diluted by the purer, and to a certain degree chemical combinations resulting. For instance, the ammonia of a polluted water may be oxidized into nitrates by the free oxygen in a purer water; or ferrous oxide, by a similar addition of oxygen, may become insoluble ferric oxide and settle to the bottom.

In addition to these processes, continual changes are being

effected by the living organisms in the water, both vegetable and animal. (Practically all reservoirs and lakes contain microscopic organisms in considerable numbers.) The former consume only the mineral matters in the water, both those originally so and those resulting from the decomposition of organic matter (except that bacteria decompose organic matter also); the animal organisms subsist upon the organized matter, including other living animal and vegetable organisms. The lower organisms have by far the greater power of multiplication, and may increase more rapidly than the higher organisms, for which they serve as food, can devour them and their death and decomposition result in a pollution of the water. As a familiar illustration, a considerable increase in mineral matter suitable for plant-food, or in the nitrogen resulting from the decomposition of organic matter, may suddenly cause the presence of vast numbers of algæ, which, not being accompanied by a similarly rapid increase in animal organisms which would consume them, cause gross pollution of the water.

" Swamps are breeding places for many of the organisms that cause trouble in water supplies, and numerous instances might be cited where organisms have developed in a swamp and have been washed down into a storage reservoir, rendering the water there almost unfit for use." (Whipple " Microscopy of Drinking Water.") The color of water in swamps is often 300 and sometimes even 500 to 700 platinum scale, and even a small percentage of swamp land on a catchment area may affect the color of the entire supply. Because of these effects of swamps, any which may be upon the area should be drained or filled in. By lowering the water level in swampy land and constructing drains around it so that clear water from above may not pass across it on its way to the reservoir, a great improvement can often be made in a supply.

If a reservoir, or portions of a reservoir, be shallow, various vegetable organisms, particularly the blue-green algæ, are likely to develop there, supported by the products of decay of organic matter at the bottom. Organic matter in shallow water may also cause the growth of aquatic plants, which may injure the quality of

the water directly by their decay, or indirectly by harboring microscopic organisms.

Organic matter in the form of trees and bushes should be removed from the bottom of a reservoir before it is flooded; and weeds should not only be all removed before the original filling, but should be cut (and removed or burned) from any shores exposed by low water before the water rises again and covers and kills them. The trees, bushes, and weeds should be cut off close to the ground. The shores should be graded if necessary so as to leave no shallow places, or depressions in which pools of water will stand when the reservoir water is lowered.

Some have advocated removing the top foot or two of soil also, since this contains roots and other organic matter. But Allen Hazen and Geo. W. Fuller, in a report in 1907 on the proposition to so strip the Ashokan and Kensico reservoirs of the New York water supply, advised as follows:

1. The stripping of the sides and bottom of a reservoir will ordinarily prevent stagnation of the bottom layers for a period of years, the length of which depends upon various local conditions. In the Boston reservoir this period does not seem to exceed ten to twenty years.

2. Ultimately, it makes comparatively little difference as to stagnation of the bottom layers whether the sides and bottom of a reservoir are stripped or not.

3. By aeration and filtration of the bottom water of deep reservoirs there can be obtained a better quality of water without the benefit of stripping, than it is possible to obtain with the aid of stripping in the absence of aeration and filtration.

4. Decolorization and purification are facilitated by the absence of stripping due to bacterial agencies which make some of the iron in the soil available as a coagulant.

5. In view of the above and as aeration and filtration will ultimately be required in order to obtain satisfactory results in this climate, present evidence and experience indicate that beyond grubbing a reservoir it is unwise to spend money for further removing organic matter from the bottom and the sides.

While water stands in a reservoir, the top surface becomes heated in summer and cooled in winter more than do the lower strata. Wind stirs the water to a depth of 5 to 20 feet, and causes it to be warmed somewhat to this depth in summer, although the warmer water, being lighter than the cold, remains always near the top. In winter the cooler surface water settles to the bottom, and the temperature thus becomes more nearly uniform, the bottom being generally somewhat the warmer.

The following table gives the average temperature of the surface of several ponds and reservoirs in Massachusetts, and of the air at the same time, by months:

TABLE No. 29

TEMPERATURE OF PONDS AND RESERVOIRS IN MASSACHUSETTS (DEGREES FAHR.)

	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
Surface of water	32 0	32 0	36 7	44 3	57 7	67 9	73 7	72 9	66 9	55 2	44 1	36.1
Bottom	39 2	39 2	38 5	42 5	43 5	43 5	43 5	43 5	43 5	43 5	44 0	40 0
Air	24 1	25 8	32 2	43 9	56 3	66 0	71 4	68 6	60 8	50 1	39 0	28.5

The higher temperature of the water is due to the sun (the air temperature being taken in the shade) and the longer retention of heat by water than by air. There were few variations from these average temperatures of more than 1° to 5°, the shallow ponds being generally the warmer.

Table No. 30 gives the temperature of Jamaica Pond (Boston) and Lake Cochituate at different depths, showing the variations referred to above.

TABLE No. 30

TEMPERATURE OF LAKES AT DIFFERENT DEPTHS

(Mass. State Bd. of Health.)

JAMAICA POND, July 14.

Depth	Surface	10′	20′	30′	35′	40′	47′
Temperature of water	75 4	75	54	42 4	42	42	41 3
Percentage of dissolved oxygen	100 (Saturated)	100 (Saturated)	49	29 47	4 18	0	0

LAKE CCCHITUATE, August 17.

Depth	Surface	10′	20′	30′	40′	45′	50′	57′
Temperature of water	74 7	66 4	53 6	49 3	48 2	48 2	45 7	44.8
Percentage of dissolved oxygen	79 15	83 69	35 86	21 33	20 93	1 65	0	0

In summer the only motion in the water of lakes and reservoirs, aside from an inappreciable current, is that due to

the wind. If the bodies of water are large and exposed, this agitation may extend to a depth of 10 or even 20 feet, where the water attains this depth; but if the body of water be small or shut in by woods there may be little of such effect felt. By this circulation oxygen is carried down by the surface water to the lower strata. It is found that all water below that which is so stirred up forms a comparatively stagnant layer. Since much of the organic matter in the water settles to the bottom, this often becomes very foul and all of the free oxygen here is utilized in nitrifying such matter. In Table No. 30 this is illustrated by data from two reservoirs; the upper 10 or 15 feet being stirred up by wind continually absorbs fresh oxygen, while retaining little organic matter to consume it; but the bottom 15 or 20 feet, containing much organic matter, is very low in oxygen. This fact is also illustrated in Table No. 31, by the difference in amount of organic matter, as represented by free ammonia, in the surface and bottom waters of several reservoirs and lakes.

TABLE NO. 31

DEPOSITS OF ORGANIC MATTER, AS FREE AMMONIA, AT THE SURFACE AND BOTTOM OF DIFFERENT BODIES OF WATER

Location.	Date.	Depth of Water.	Depth of Deepest Sample.	Free Ammonia.	
				Surface.	Near Bottom.
Jamaica Pond, Mass....	Aug. 14	57	50	0.0000	0 4720
Waban Lake, Mass..........	Aug. 27	36	35	0.0012	0 1760
Lake Cochituate, Mass.......	Sept. 18	65	60	0 0004	0.0680
Wenham Lake, Mass.........	July 24	46	45	0.0000	0 0560
Boston Reservoir No. 4.... ...	Aug. 31	46	40	0.0000	0 0012
Lake Winnepesaukee, N. H...	Aug. 28	. ....	110	0.0000	0 0000

This table also shows, by the last two illustrations, that the stagnant layer is not necessarily foul, but only when organic matter is present in the water.

"As the surface-water cools in the autumn and becomes heavier than the water below the surface, vertical currents are

produced which extend down to and somewhat beyond the depth where the water is at the same temperature as at the surface. These currents are nearly continuous and extend deeper and deeper as the season advances, until some time in November, when they extend to the bottom of the pond. After they have reached the bottom they continue to keep the water in motion for several weeks until the whole of the water in the pond has reached the temperature of maximum density." "In a lake with any considerable amount of organic matter in it and also in deep artificial storage reservoirs, where the surface has not been stripped, the lower layers, which are quiescent during the stagnation period, gradually collect all the organic matter from the upper layers, and decay goes on until the oxygen is used up. The water becomes darker and darker, until by October it is very yellow, and generally has a disagreeable smell. Of course, when the great overturning comes, in November, all this bad water is brought to the surface, and the infusoria and diatoms begin to grow in enormous numbers, because the organic matter and oxygen are brought together and provide food for organic life. The same phenomenon takes place in the spring period of circulation, although on a smaller scale." (FitzGerald on the "Temperature of Lakes," Trans. Am. Soc. C.E., Vol. XXXIV.)

In Fig. 23 are shown the typical winter and summer temperatures of a lake that freezes.

The above explains the sudden presence in water supplies of unpleasant tastes due to algæ, as stated in Art. 9.

If the water-supply be drawn from the surface of a reservoir from May to September, the purest water will thus be obtained. If now, just before the overturn, the bottom layer of impure water be drawn off through a waste pipe, much of the fouling of the reservoir will be avoided. If neither the water nor the bottom of the lake or reservoir contain organic matter or nitrogen, all this trouble is of course avoided.

If much unoxidized organic matter remain in a reservoir when this is frozen over, and the access of additional oxygen to the water from the air is thus shut off, putrefaction may

take place with its resulting gases; but this can happen only when the water is more impure than any supply should be.

Ice is not an important source of supply in this country, although it is in some extreme northern ones, and in certain localities in the Alps. A comparatively small amount is used in ice water, however, and for this reason its purity is of importance. Impure ice is as dangerous as impure water, and is more commonly found in use as a beverage; some families using melted ice in summer as their principal drinking-water, which ice may have been obtained from a highly polluted

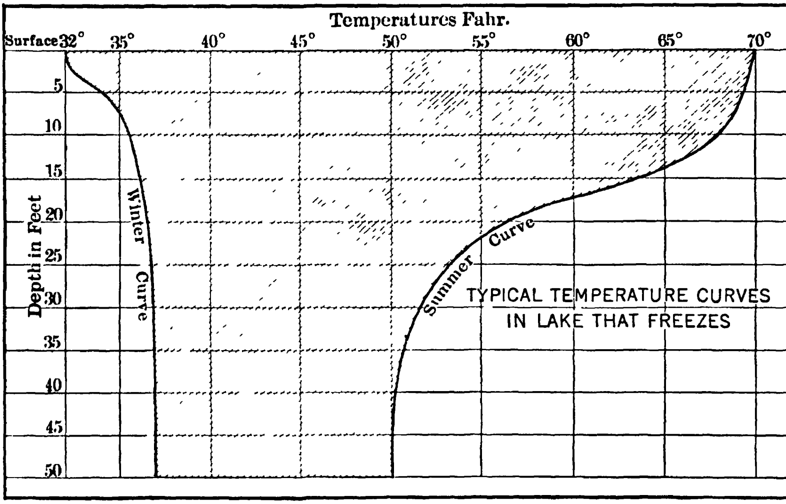


FIG. 23.—Temperature of a Lake that Freezes.
(From Trans. Am. Soc. C.E., Vol. XXXIV.)

pond. The popular idea that water is purified in freezing is partially true, but much of both organic and inorganic impurity frequently remains in the ice. When water is frozen slowly, however, much of the impurity is excluded and is taken up by the remaining water, which thus becomes more impure. Hence the most impure ice is that frozen last, and is generally found at the bottom, if from a shallow pond, or at the center of artificially frozen cakes. From deep ponds, however, the under side of the ice is purest, because the slight increase in impurity of the lower water caused by the freezing of the surface is more

than offset by the greater percentage of purification effected by the slower freezing of the under ice. When ice is flooded, all the impurities in the flooding water must be contained in the ice. This refers to bacteria as well as to other impurities. Dr. Prudden found in ice which had been frozen for eleven days 1,019,403 bacteria per cubic centimeter; in that frozen seventy-seven days, 72,930; and in that frozen one hundred and three days, 7348 per cubic centimeter. He also found from 50 to 200 times as many bacteria in snow- or bubbly-ice as in clear, transparent ice free from air.

CHAPTER VIII

RIVERS AND LAKES

ART. 39. QUANTITY OF RIVER WATER

SURFACE water is generally collected as well up toward the head of a stream as possible, both because the greatest fall is thus obtained to the point of utilization, and because here are more frequently found the best locations for reservoirs and dams. This source of supply is most applicable to hilly or mountainous country, whose watersheds are sparsely occupied—conditions generally associated with a poor and thin soil or slopes too steep to be cultivated.

In the lower lands there are few locations for reservoirs, and pumping must generally be resorted to; moreover the streams here are usually of sufficient size to furnish an ample supply even in dry times. The water is therefore taken directly from the stream, and no storage is required, although it may be desirable for permitting sedimentation.

The quantity of water flowing in a river is that reaching it by surface or underground flow from all the drainage areas above on all its branches, less what may have been removed by evaporation, by seepage, and by man for irrigation and other purposes. A part of the flow may in some cases be underground, beneath and near the bed of the river.

The yield to the river will be the total yield of the drainage areas of all its tributaries, the estimating of which has already been discussed under the head of surface water. The seepage may vary from almost nothing to the entire volume of flow, depending upon the character of soil, amount and character of sediment carried by the stream, and height of ground water. In a clay or rock channel the seepage will be very small. In a sandy soil it may be great; but if loamy or clayey matter is carried to the stream by heavy rains, this will gradually be

deposited as sediment upon the bottom and form a nearly impervious channel.

If the ground water stands level with or above the river surface, there will be no loss by seepage, but rather a gain. Such ground water, however, must be derived from the drainage area of the stream, and hence be included in the general calculation of yield. This ground water will usually flow slowly both towards the river and down its valley.

In many Western rivers the flow is in places altogether beneath the surface during a large part of the year, the river bed being dry except during rainy seasons. At certain points in the courses of many of these rivers, rock or clay outcrops through the porous soil, and here the water is forced to the surface and flows in the channel, to disappear again further on where the impervious stratum again dips beneath the surface. Several of these rivers have no visible outlet, but simply disappear into the ground, from which the water is absorbed by vegetation and evaporated. A few rivers flow for a part of their course through underground caverns, generally in limestone.

The underground flow of a river can either be utilized as ground water (see Chapter IX), or in some cases can be intercepted by a dam carried down to the impervious stratum and across the channel of pervious soil which affords it passage. Such a dam, which causes the porous soil above to act as a storage reservoir, was built across Pacoima creek, Cal., the channel in bed-rock being at this place but 550 feet wide and filled with gravel to a depth of 40 feet.

An estimate of the quantity of water flowing in a river at various seasons can be made by deducting from the rainfall upon its entire catchment area the evaporation from the same area both from earth (including plant consumption) and from water surfaces; or by other methods of estimating yield referred to in Art. 35. But the only accurate method is by direct measurement, and this is of greater value the longer the series of years it covers. Variations in river flow are illustrated in Tables Nos. 26 and 27, and it is here seen that the maximum may be

500 times the minimum, and the latter but 5 to 10 per cent of the mean annual flow. If this minimum amount is not more than the consumption, an additional source must be obtained; or means provided for storing the river water, either by a dam in the river itself, or by a storage reservoir into which the water is pumped. The latter is generally preferable, but frequently the more expensive of the two.

Art. 40. Quality of River Water

The quality of rain water when it reaches a stream has already been considered, but there are many changes in it continually taking place after this. As in the case of reservoirs, mineral and other impurities carried in suspension are deposited as the current velocity becomes less, but more slowly in rivers because of the greater motion in the water. The stratification found in reservoirs and lakes does not exist in rivers, the current keeping the water in constant circulation. For the same reason the temperature of rivers is more uniform at different depths, although varying more from month to month, the variation being between 32° and 80° in seven Massachusetts rivers. Also more oxygen is generally available for nitrification in a river than in a lake or reservoir, since all parts of the water are in turn brought in contact with the air.

In spite of these means of purification, however, the water of a river is generally less pure than that of the run-off contributing to it, owing to the impurities reaching it from the various farms and communities past which it flows. The most dangerous of these is sewage contamination, although that from slaughter-houses and rendering establishments is fully as offensive and is far from being harmless. Waste waters from dye-works and numerous other manufacturing industries may render a water totally unfit to drink. A minor source of impurity, although it may become an important one, is the waste from passenger-steamers and other boats.

The pollution from manufacturing establishments may consist of almost any acids, alkalis, or organic matters. A carpet, blanket, and cloth mill on the Schuylkill river used

daily, a few years ago, 48,700 pounds of organic matter in 18 different forms; 2520 pounds of 21 different acids; and 950 pounds of 6 different alkalis. Brass works discharge considerable sulphate of copper, cyanide of potash, and oils. The principal waste from iron works is sulphate of iron; from paper mills come filaments of jute, cotton, and other organic matters, caustic soda, chloride of lime, and sulphite; from woollen factories the washing of the wool produces large amounts of organic wastes, and soda, alkalis, logwood, fustic, madder, copperas, potash, alum, blue vitrol, muriate of tin, and other dye wastes are found in the waste waters. This list might be continued indefinitely; but the appearance of most rivers receiving such wastes is evidence of the seriousness of the contamination.

The following table gives an analysis of the Passaic river, showing gross pollution due partly to manufacturing wastes, but even more to sewage pollution; and also an analysis of the relatively pure Hudson, although this receives the sewage of several cities and towns above Albany.

TABLE No. 32

ANALYSES OF PASSAIC AND HUDSON RIVER WATER

Parts per 100,000.

	Ammonia			Nitrogen as						
	Free.	Albumin.	Chlorine.	Nitrites.	Nitrates.	Total Solids.	Loss on Ignition	Mineral Matter	Oxygen Required	Bacteria per c c.
Passaic, below Passaic Falls.	02947	0422	8116	0342	00296	10 856	3 831	7 025	.5694	747,000
Hudson, above Albany..	0030	0087	35	trace	000	7 3	3 5	3 8	376	

Although there is considerable movement of river water, both vertically and across the stream, still the greater part of the suspended material transported, together with the bacteria, is found near the bottom; also an impure stream entering on one side of a river may travel for miles before being equally commingled with the purer water. These facts should be taken advantage of in locating an intake.

A great objection to many river waters is the large amount

of mineral matter in suspension carried in time of flood. The use of turbid water is in some cases avoided by providing storage reservoirs holding sufficient clear water to permit the discontinuance of pumping when the river is muddiest. This is always after a rain, and may last for but a day or two at a time, the duration varying directly with the size of the drainage area above. The extreme variation in the amount of silt present in some rivers is illustrated by the Ohio, in which during one year the maximum amount of suspended matter found was 531.1 parts per 100,000, the minimum was 0.1 part, and the mean 22.5 parts.

It is probable that manufacturing wastes and sewage are in most cases quite constant in amount, and hence the polluted water is most impure when the river is low. The quantity of organic matter washed from the catchment area, which may include considerable human excreta, will be greatest after a rain. There will in most rivers be a wide variation and sudden changes in the impurities found, both mineral and organic. Table No. 33 shows such variation for the Hudson river above any direct sewage inflow.

TABLE No. 33

VARYING AMOUNTS OF IMPURITIES IN HUDSON RIVER WATER (MASON).

(In parts per 1,000,000)

Date.	Ammonia.		Nitrogen as		Chlorine.	Required Oxygen	Total Residue.	Loss on Ignition.	Suspended Matter (Silt).	Temperature, Degrees Fahr.
	Free.	Albuminoid.	Nitrites.	Nitrates.						
Nov. 3..	.030	.087	.000	trace	3 5	3 76	73	35	88 4	
Dec. 15.	.045	.150	trace	.15	4 5	13 00	107	42	68 6	36
Jan. 12..	.025	.080	.000	.10	3 5	7 65	43	39	0 0	
Feb. 5...	.055	.100	.000	.15	....	8 85	88	45	0 0	34 6
Mar. 4..	.085	.150	trace	.10	....	10 00	93	51	0 0	33.0
April 5..	.042	.235	trace	30	2 4	5 90	388	88	11 0	46 4
April 10	.058	.660	trace	trace	....	15.50	583	74	495 0	41.0
May 8..	.030	.205	trace	.10	2 5	7.30	67	31	0.0	68 0
June 5..	.045	.120	.000	trace	3 5	8.70	78	50	0 0	71 0
Sept. 20	.280	.320	.0015	30	5 0	2 65				
Oct. 30..	.055	.155	trace	....	3 5	14.85	101	57	0 0	44

The record of April 10th shows the largest amount of albuminoid ammonia coincident with that of suspended matter, and hence probably caused by rain.

The increase of bacteria caused by rain washing them into a stream is illustrated by the Croton (New York City) water, which ordinarily contains about 35 bacteria per cubic centimeter, but after a hard rain as many as 7200 have been found.

ART. 41. LAKES

A lake is generally but the broadening of the channel of a river or other stream, and the water entering a lake is but that of such stream. The quantity of water passing through a lake is no more than that flowing in its river; but when this latter becomes temporarily small in times of drought the lake acts as a storage reservoir, and hence is generally preferable to a river as a source of supply, if the quality is equally as good.

In passing through a lake, water often undergoes changes in quality which would not occur in the stream. Lakes are ordinarily found in a hilly country, where the currents of the streams are more or less rapid, while that through the lake is very slow. Suspended matter which was carried by the stream is therefore permitted in a lake to settle to the bottom, and the water is thus clarified, many bacteria being carried down during the sedimentation, or dying off from lack of food-matter. The water at the lower end of a lake is hence in most cases purer than that at the upper end, provided no pollution finds its way into the lake from the shores. There being no more water flowing through a lake than flows in the river in whose course it lies, there will be in the long run no more dilution of sewage or other impure water discharged into a lake than if the same were discharged into the river; but the water may become more pure in passing over a given distance, because of the greater time occupied and the greater opportunity for sedimentation thus afforded.

Most lakes are deeper than their rivers, and such effect as

depth may have upon the quality of water is found in many or most lakes. Owing to this depth, to the size of a lake as compared with a river channel, and to the slight current movement, lakes offer better opportunities for locating water-works inlets than do rivers or smaller streams.

Like river water, lake water must ordinarily be pumped; except in the case of lakes on mountain streams, which act practically as natural reservoirs of surface water. In fact, lakes and reservoirs have in most respects similar effects upon the quality of water; and most of the statements made in Art. 37 concerning reservoirs are applicable likewise to lakes.

CHAPTER IX

GROUND WATER

ART. 42. WATER-BEARING STRATA

THE rainfall absorbed by the soil of each catchment area, after percolating downward, continues to travel in some generally horizontal direction toward a stream, lake, or sea. Its underground passage is subject to many of the laws affecting surface flow—its surface must fall in the direction of flow, and the velocity of flow is proportional to this fall; the hydraulic gradient of this flow cannot be, at any point, lower than the body of water into which the flow discharges; the water will seek the lowest accessible channels, but ordinarily fills the soil over large areas. There are the additional influences of friction in passing through the soil; the capillary attraction of this; and confined flow caused by super-strata of impervious material, the conditions then approximating those found in water pipes or other conduits under pressure.

The amount of flow is dependent upon the size and perviousness of the catchment areas which contribute to it, and upon the precipitation upon those areas. It is in many cases, however, increased by seepage from rivers crossing the pervious stratum, the drainage areas of which rivers are not strictly parts of the catchment-basin in which this pervious stratum lies.

If the water-bearing stratum extends to the surface of the ground, it may occupy a pocket in an impervious under-stratum of clay or rock, in which case water permeating it is held until evaporated or withdrawn by wells. Swamps are formed by such conditions when the amount of water reaching the pocket suffices. In more cases a surface pervious stratum is a sedimentary deposit made by a stream, probably combined with talus and

surface wash from hills paralleling the stream; and this stratum has an outlet into the stream and also a general fall paralleling that of the stream. In most cases there is a humus covering to such pervious deposits which is much less pervious than the material below. These deposits are filled partly by seepage from the river, and especially from its flood waters when they stand for a few days over and soak into the deposit from the entire surface; which water drains out slowly after the river recedes; this draining not being completed, when the deposit is fine-grained, before the next annual flood. These deposits are also supplied with water by underground flow (and more or less surface flow) from

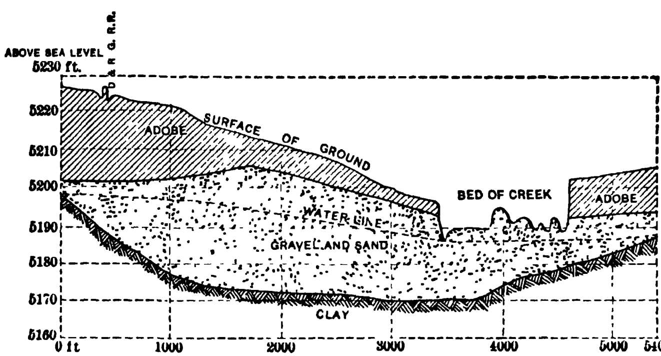


FIG. 24.—Cross-section of Valley of the Fountain qui Bouille, Pueblo, Colo.

the hillsides toward the river. All of the water moves continually toward the stream and tends to seep into it until it falls to the normal level of the water in the stream. It also has some motion parallel with the stream, the amount of this depending upon the relative slope of the ground water normal to the line of the stream as compared with the slope of the stream itself. If the pervious material underlies the stream, as is often the case, there is practically an underground stream as well as one visible on the surface, and the former may continue after the latter has " run dry." The underground flow in some cases is known to follow an old stream bed in the underlying rock which is 100 feet or more below the surface and may only approximately follow the present course of the stream.

In the Northern states moraines of gravel and sand furnish pervious receptacles of water which is discharged as springs at the foot of their lower slopes, as seepage into streams, etc. These generally derive their supply from precipitation directly upon them and from the run-off of higher land abutting them.

In rugged mountainous country alluvial deposits are often found at the mouths of gorges, generally spreading out fan-shaped, sometimes several miles in extent and up to 2000 or 3000 feet thick. Such a deposit the stream from the gorge supplies with water, which seeps through it and emerges along its foot.

A stratum of sand or gravel may be overlaid with clay or other impervious material, or pervious sandstone with impervious rock,

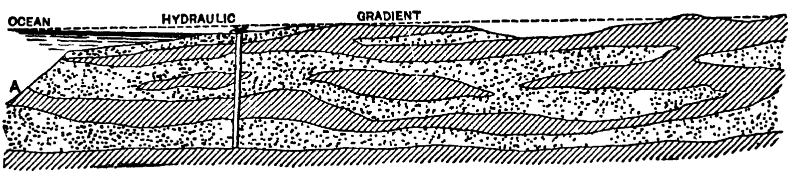


FIG. 25.—Typical Section in Central New Jersey.

and the pervious stratum may reach the surface and there receive a supply of ground water by direct precipitation, run-off from adjacent higher land, or a stream that flows over it, or water may enter it through breaks or faults in the impervious strata that lie above it. This bed of pervious material may have no outlet, and the water stand in it under pressure; but in most cases it has an outlet into the ocean or other deep body of water, or outcrops at the surface at another point.

In many sections of the country the several strata to the depth of thousands of feet have been depressed into a basin or trough form, and the pervious strata outcrop in a ring or in two parallel strips with the depression between them. The exposed outcrops take in water in the ways before described, and this settles into the depression, where it is held under a hydrostatic head by the impervious strata above. If the strata are bent into the shape of a trough, the ground water flows in the direction of the slope of the trough; but the friction of flow in this

will ordinarily produce a considerable hydrostatic head except near the mouth of the trough.

If a break or fault exist in the impervious stratum over a water-bearing one under hydrostatic head, or a well or other hole,

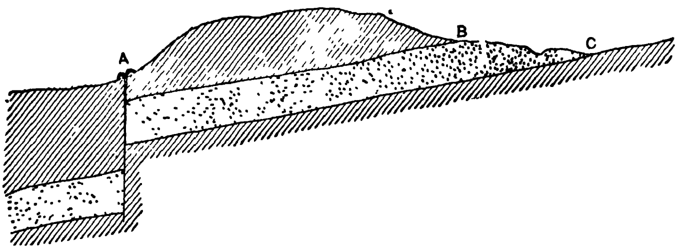


FIG. 26.—Spring from a Fault.

the water will rise in this to the hydraulic gradient, if the opening extend this high. If such hydraulic gradient lie above the ground surface, the water will rise to the surface. If a well be driven at such a point, it will be a flowing or "artesian" well,

FIG. 27.—Ideal Section Illustrating Condition Causing Artesian Wells.

and the elevation of the hydraulic gradient above the ground is called the "artesian head." If the opening in the impervious stratum extend only to an overlying pervious one, water will rise from the lower to the upper pervious stratum and enter

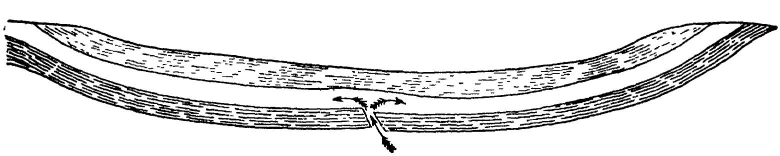


FIG 28.—Ideal Section of Artesian Basin Charged with Water from Below through Fissure.

the latter, or will settle from the upper to the lower, depending upon the relative hydrostatic pressure in the two.

The temperature of the ground increases with the depth about 1° F. for each 60 feet, or possibly less in some localities; this

relation not applying definitely to the upper 50 feet, however, because of the effect of seasonal and other surface temperatures to about that depth. Therefore water that rises from considerable depth may be warm or even hot. (Ground water is called "warm" if its temperature lie between 60° and 100° F., and "hot" if it be more than 100°). The depth from which a

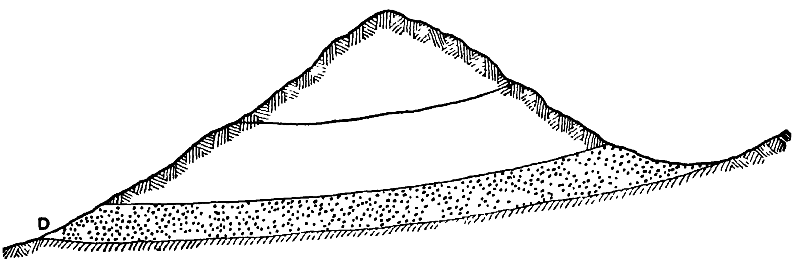


FIG. 29.—Spring at Outcrop.

water rises may therefore be determined approximately by its temperature; although if it rise slowly through other strata rather than in a well, it will probably cool more or less in the meantime. As stated above, water may rise from one deep stratum to a higher one through a fault or fissure, and thus the water tapped by a well or emerging as a spring from this stratum will have a higher temperature than that due to its depth. Where the geological structure suggests an artesian basin, the existence of warm springs therein is an assurance that artesian wells can be developed there; and the temperature will give an indication of the depth required for such well.

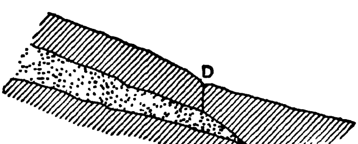


FIG. 30.—Spring in Hardpan.

All rocks contain small amounts of water, but except for some sandstones the amounts are so infinitesimal as to be negligible for even domestic water supplies. All rocks, however, are traversed by numerous fissures or joint cracks, and water finds its way into and along there. Limestones and a few other rocks are dissolved by weakly acid water, and in these the fissures

are widened into sink holes, subterranean passages and caves. Streams, large or small, may find their way into these and flow as underground rivers; but the water here, of course, is not under pressure, and generally emerges at lower levels in the form of large springs.

Fig. 24 illustrates water-bearing gravel or sand in a river valley in which the underground flow exceeds that in the creek during most of the time, and is more constant in volume; and in many western rivers the underground flow is more relied upon for supply than the visible flow. In Pacoima creek a dam was built

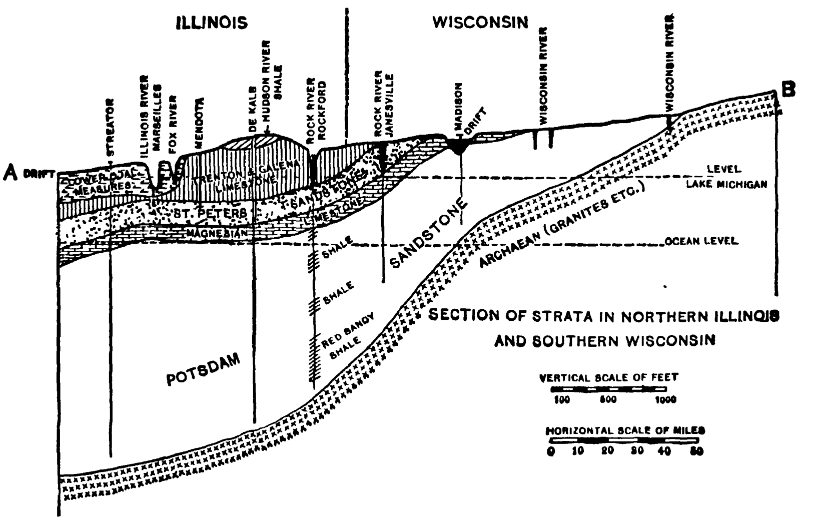


FIG. 31.—Artesian Wells in Illinois and Wisconsin.

across the valley, down to the impervious bottom, to intercept all such flow; but ordinarily this is uneconomical because of the great length and depth of dam required and the relatively small flow intercepted.

Fig. 27 is an ideal section of a dished stratum producing artesian conditions; and Fig. 28 of a fissure in the underlying impervious stratum through which water from a deeper water-bearing stratum rises into a higher one. Fig. 31 is a section passing through Streator, Ill., and Madison, Wis., showing the outcropping of the Potsdam and St. Peters sandstone,

which furnish water to a large number of cities in the north central part of the United States. The water rises to the surface in a large number of wells in the Potsdam and a few in the St. Peters sandstone. The former has an area of outcrop in the upper Mississippi valley of about 14,000 square miles. In Ohio and Indiana the Niagara and Trenton limestones furnish abundant supplies; and in the Dakotas and northern Nebraska the Dakota sandstone.

If *AB*, Fig. 32, is a stratum of sand or pervious sandstone between two strata of impervious clay or rock, the percolation from the catchment-basin or valley *B* will travel toward *A* and emerge there. The hydraulic gradient will be the line *BC*,

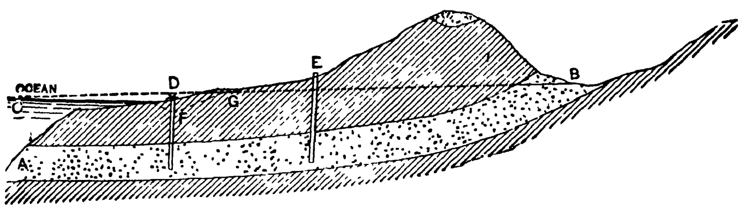


FIG. 32.—Underground Flow in Stratified Rock.

which will be straight if the material and thickness of the stratum *AB* be constant. A well at *D* would then overflow at any point below *BC*; while in that at *E* the water would rise to this line only and would need to be pumped. If water were drawn from *D*, the hydraulic gradient would be lowered at that point. If lowered to the point *F*, it is probable that some salt water would reach the well, as happened at Galveston, Tex. The end of the hydraulic gradient *C* is mean tide level, friction in the pervious material preventing the changing pressure at *A* due to tides from having any effect for more than a few hundred feet inland from *A*.

The amount of water emerging at *A* is in some places so great as to occasion fresh-water springs in the ocean; as off the coast of South Carolina, and the Gulf coast of Florida, where such springs boil up through a depth of 100 to 300 feet of water in such volume as to make it difficult to row a boat above them.

In New Jersey, south of the Raritan river, the strata for several hundred feet down are blue and yellow clays and marls, alternating with strata of sand of varying thickness. Most of the sand strata are water-bearing. The flow from a depth of 425 feet in one well half a mile from shore rose to $6\frac{1}{2}$ feet above mean high tide, indicating an outlet into the ocean at least 8 or 10 miles distant (as at *A*, Fig. 25). Similar conditions are found along the Gulf of Mexico and over a considerable part of the South Atlantic coast.

Somewhat similar conditions, except that the water-bearing strata do not emerge below a body of water, illustrated in Fig. 29, produce springs or strips of swampy places; although both of these are more frequently due largely or wholly to shallow ground flow from the land immediately above, which is forced to the surface by the outcropping of an impervious stratum.

Springs are also caused by water rising through faults, as at *A* in Fig. 26. Such a spring at San Antonio, Tex., yielded 50,000,000 gallons daily. Underground water may also rise through a stratum of clay or hardpan, as in Fig. 30.

Water in valley bottoms (Fig. 24) furnishes the supply to a large number of cities in this country. The alluvial deposits in the southern and central valleys and the drift in the northern third of the United States afford many abundant supplies of this class; most of which, however, are being or should be abandoned on account of the danger of pollution. Shallow wells and galleries are constructed for utilizing this source of supply.

Water-bearing sand strata alternating with clay are probably used more extensively for water supplies than any other class of underground sources. The Atlantic coast states from New Jersey to Georgia, and the entire Gulf coast, offer this source of supply in the alluvial and marine deposits at depths of 50 to 1000 feet. Pensacola, Fla., obtains from this source 2,000,000 gallons daily; Memphis, Tenn., 15,000,000 gallons; Brooklyn, N. Y., 150,000,000 gallons; Fort Wayne, Ind., 6,000,000 gallons. In western Florida, Mississippi, and Louisiana water

is found in abundance in sand and fine gravel interspersed with strata of vari-colored clays. West of this around the Mississippi river silt predominates and the wells yield scantily. In eastern Florida the wells penetrate cavernous limestone, spurs of the Georgia mountains. North of this are again water-bearing sand strata. A typical Gulf section is shown in Fig. 33, running north and south through Pensacola. Here the hydraulic gradient falls about 1 in 4500 ($s = .0002+$), the water rising 16 or 17 feet above sea level. At Natchez, Miss., the water-bearing sand stratum is but 40 feet thick.

The impervious strata are generally clays and marls, with occasionally hardpan, and vary in thickness from 5 to 50 feet

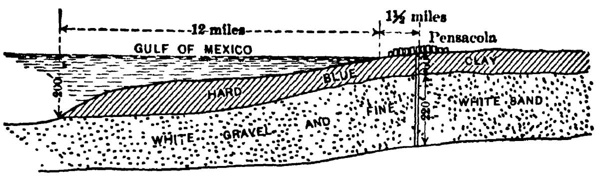


FIG. 33.—Section of Water-bearing Stratum at Pensacola, Fla.

or more. The pervious material, generally sand, sometimes runs in streaks in irregular courses as in Fig. 25, and again spreads out into thick strata miles in width. There can therefore be no certainty of finding a particular stratum by boring at any given point, but this is largely a matter of chance. The thicker and more important water-bearing strata generally extend over large areas, however, and may be found with considerable certainty at any point within a district whose outer limits it is known to underlie.

ART. 43. FLOW OF GROUND WATER

The general direction of flow of ground waters discharging into the ocean is, in the majority of cases, towards the ocean. It is along the line of glacial motion if in drift, or diagonally across and down a valley if in alluvial deposits.

The velocity of flow depends upon the slope of the water surface; the size and uniformity of the grains of sand or gravel,

or of the pores of the rock through which it flows; and upon the temperature, although this ordinarily varies but little. In finer gravel and sand the velocity is found to be directly proportional to the slope, while in coarse gravel it is more nearly proportional to the square root of this. The velocity in sand may be represented by the formula

$$V = cd^2s,*$$

in which V = velocity in feet per second;

c = a coefficient, about 0.29 as determined by a few experiments by Hazen; but 0.45 as indicated by later experiments.

d = the effective size of the sand grains in millimeters. (Effective size of sand "is such that 10 per cent of the material is of smaller grains and 90 per cent is of larger grains than the size given.")

$s = \frac{h}{l}$ = sine of the slope of the hydraulic gradient.

This formula is not considered applicable when d exceeds 3.

The relative area of open spaces in sandy soil through which the water flows determines the quantity of flow. In a given cross-section this area will generally range between .35 and .60 of the total area. The quantity of flow, Q, per unit area of vertical section would then be $.35V$ to $.60\,V$. On Long Island, where the Brooklyn water supply is obtained, s is about .0002 in dry weather to .002 in wet. This would give a velocity of flow in dry seasons, assuming $d = 0.5$, of $V = 0.45 \times .25 \times .0002 = .000022$ feet per second, or 1.88 feet per day; and $Q = .40 \times 1.88 = 0.75$ cubic feet per day per square foot of vertical section. In wet seasons these values might be ten times as great, s being much greater.

Slopes in sand of 30 to 50 feet per mile are found ($s = .0057$ to .0091), the slope generally increasing with the fineness of the material. In valleys having gravelly soils the cross-slope is generally very flat, while the longitudinal slope is practically that of the river.

* Allen Hazen, in Report of Mass. Bd. of Health.

Through rock the velocity of flow is less than through sand, owing to the presence of the interstitial cementing material, but practically nothing definite is known upon this subject. In the Dakota sandstone the distance from the outlet in the Missouri river to the catchment outcrop in the Rocky mountains is about 500 miles, and the difference in elevation about 5000 feet, giving an average value of *s* of .002; but it is thought that the hydraulic gradient is steeper than this near the outcrop, since it is flatter in Nebraska; probably because of great irregularities and faults in the strata in and near the mountains.

If the quantity of flow through a given material due to different water slopes is known, we can approximate the amount of water available from such material in a given locality by sinking two wells in the line of flow, but some distance apart, and noting the water level in each, thus determining *s*; or, if the direction of flow is not known, both this and *s* can be determined by sinking three wells at approximately the corners of an equilateral triangle. The velocity of flow can be found approximately by noting the time elapsing after placing salt or a dye in one well before it makes its presence known in another directly below it in the line of flow. If the upper well is artesian, rock salt may be lowered to the bottom of the well in a bag and the flow from the well be immediately stopped.

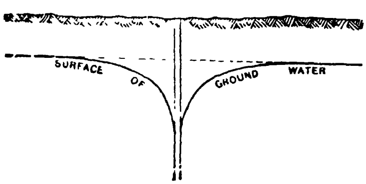


FIG. 34.—Curves Showing Cone of Depression.

When a well is pumped, water flows to it from all sides. If we imagine a series of vertical cylindrical surfaces concentric about the well at different distances from it, the same amount of water must pass each cylinder; but as their circumferences vary in length as their radii, the velocity of flow per foot of circumference will be inversely as the distance from the well. The greater velocities require greater slope in the ground-water surface, and this therefore assumes the shape of a cone of depression with curved elements, as in Fig. 34.

This still further reduces the area of the cylindrical surfaces nearer the well by decreasing their depths, thus making the velocity of flow increase more rapidly than the radii decrease. The depth and area of the depression at the well therefore increase in a proportion equal to or somewhat greater than the amount of water pumped from it; the shape of the curve of depression depending upon the slope at each point, which may be obtained theoretically by solving for s in the formula $V=cd^2s$. A Brooklyn, N. Y., well, when being pumped, showed a depression of 6

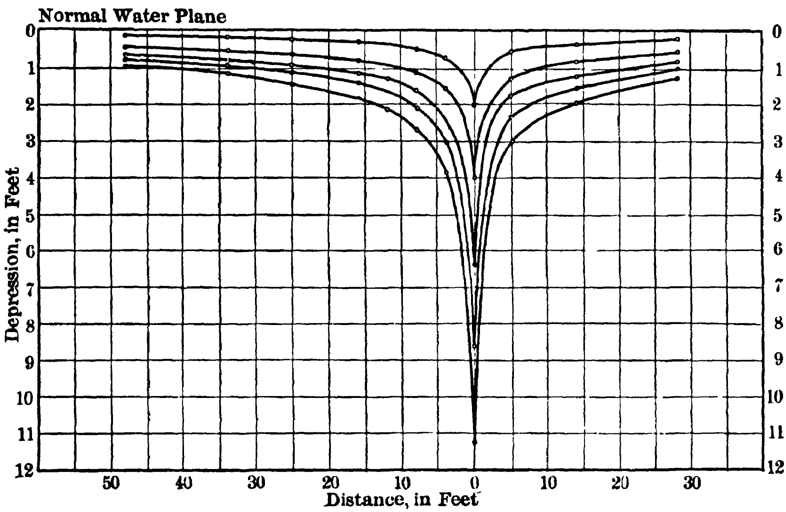


FIG. 35.—Curves Showing Depression in Water Plane near 15-inch Pumped Well in Arkansas River Valley.

inches at 4300 feet from the well, 26 inches at 2300 feet, and 56 inches at 300 feet. In Parkersburg, W. Va., the cone was about 400 feet in diameter when the water was lowered 13 feet. In a Brooklyn well the diameter of the cone was about 5000 feet with a 15-foot depression at the well. In the well shown in Fig. 35 the yield was 14½ gallons per minute for each foot of depression below the normal water plane. In Memphis the curve of depression was fairly well represented by $x=\frac{2785}{y^{0.58}+50}$.

Some time must elapse after pumping begins before the depression assumes a final shape, since the water originally

filling the cone must first be exhausted. Similarly, the depression will fill gradually when pumping ceases. Thus, a set of wells in Brooklyn was pumped for twenty days, at a rate of 5,000,000 gallons per day, the ground water being lowered 14 feet; and the depression was not filled until the twelfth day after pumping ceased. If the diameter of the depression was 5000 feet, and the soil was 33 per cent voids, and 60 per cent of the contained water was yielded, the amount of water to be replaced was probably about 95,000,000 gallons—approximately the total amount pumped. If this was the case, the depression might not have assumed its permanent form when pumping stopped.

If wells be located too near each other, their cones of depression will intersect, and the flow of each or some will be reduced. They should generally be placed across the direction of flow, otherwise the upper wells may diminish the flow of the lower; but water flows to the wells from sides as well as above, and if the cones of depression do not intersect, the interference of the upper wells will probably not be serious.

Art. 44. Utilizing Ground Water

Ground water may be brought into use through the medium of wells or infiltration galleries, or as it issues in the form of springs. The last is the cheapest where the water issues in large quantities at single points; but where it appears on the surface as a more or less general exuding over a large area, the collecting, including cost of the required area of land, may become quite large. Certain large springs have been dug out and a basin walled around them, and thus serve as supplies for small municipalities; but the number of these is few. Where ground water flows through linestone caverns, the intercepting of it where it emerges (generally in a large volume) is often the only method of utilizing it.

In many cases a spring supply may be increased by concentrating at one point a flow that naturally exudes over a considerable area. A trench, either open or filled with gravel, or a drain,

placed across the line of flow and carried down into the impervious stratum on which the water-bearing stratum rests, may serve this purpose. Or a lowering of the outlet of the spring may draw from other channels that drain the same catchment area. A spring yielding 75,000 gallons per day was so developed in this manner by the author as to yield more than double this amount and furnish the supply for a small community. If there are one or more porous strata near and parallel to the surface of a hillside, horizontal tunnels may be driven into the hill to intercept water from these, as was done at Oakland, Cal.

Wells may be dug, bored, drilled or driven. The last three consist of pipes sunk into the ground to depths varying from 5 or 6 feet to as many thousand, the diameter varying from 1½ or 2 inches for the shallower wells to 15 or 18 inches for the deeper ones. The shallowest wells draw their supply from surface deposits, which are subject to considerable fluctuations in yield and also to pollution from stables or privies near by. If a driven well pierces an impervious stratum of considerable extent, the water drawn is pretty certain to be safe from surface pollution. But thin clay strata may terminate near a well (as in Fig. 25), of which termination there is no surface indication, and care should be taken to investigate this. Deep wells generally pass through several strata of various materials. The deeper they are, as a general thing, the more distant is the source of the ground water, the more constant the supply and the more certain to be free from any matters other than dissolved minerals.

Water enters wells through holes or slots in their walls, and the larger the total area of openings in a well the greater the amount of water obtainable. But the openings in any one well should not, as a general thing, connect with two distinct water-bearing strata, since this would permit the water under the higher head to escape into the stratum where the head is lower. If it is desired to use water from two or more strata, a separate well should be sunk to each. For the same reason a well casing should fit tightly all impervious strata through which it passes.

In order that all surface water (which may carry pollution)

may be excluded, the well casing should be tight from the surface to a thick impervious stratum of clay or rock, with which it should make a tight joint; or, if there are no such strata, the entire casing should be tight except near the bottom. In every case the top should be tightly closed or else carried above the reach of surface water.

If a well is not artesian, it is of course necessary to raise the water from it by some method of pumping. Even if it is a flowing artesian, if the amount of water so obtained is not sufficient, it is generally cheaper to pump from it rather than sink another well. By pumping, the level of the hydraulic gradient at the well is lowered and water is drawn to the well from a considerable radius around it and the amount may be increased several fold. No more can be obtained, however, than will enter through the openings in the casing below the lowered ground-water level. With any practicable number of wells it probably is impossible to obtain more than a fourth or a third of the underground flow.

For a short time after beginning to pump a well, the delivery may increase, owing to the opening of channels in the water-bearing stratum by removal therefrom of clay or fine sand. After this has been accomplished, the amount yielded should remain constant while the depth in the well is constant, and should vary approximately as the depth of water surface in the well is lowered below the original ground-water level. If, while the level in the well remains constant, the flow decreases, it indicates that the draught is exceeding the natural yield or else that the openings in the well casing are stopping up with fine sand or other material. If the former is the case, the delivery will continue to decrease unless the rate of pumping be reduced to the point where the volume remains constant under a constant well-water level. If a series of wells over a considerable area exhibit this falling off, it indicates that the ground water available for that district is being overdrawn, and the draught should be diminished. At Rockford, Ill., such overdrawing by a number of wells in the Potsdam sandstone lowered the ground-water level 10 feet in six years. To prevent the useless lowering of water in an artesian basin, local laws should require that

every well or drill hole be cased to prevent water escaping from one stratum to another; that every flowing well when not in use be securely closed and every abandoned well be filled with impervious material to prevent waste of the water.

Dug wells are seldom carried to a greater depth than 50 feet, and are generally used for obtaining water from a surface deposit. They serve as open reservoirs for collecting ground water. from which it is pumped. The advantage of these is that greater amounts of water can enter than can pass the openings in one or several driven-well casings. They are therefore useful especially where the supply of water is abundant and the water-bearing stratum very porous. In some cases a dug well is sunk to a depth of 25 or 50 feet, and driven wells are sunk from the bottom of the dug well and pumps placed on the floor of the latter, which thus serves really as an underground pump chamber.

Infiltration Galleries. When the ground flow near the surface is to be used for a supply, a number of wells, driven or dug, may be employed; but a larger quantity can ordinarily be obtained by use of a long crib placed at right angles to the direction of flow and below the ground-water surface. This is generally placed near a river, not so much to utilize the river water, but because the ground flow increases in volume as its outlet is approached. Infiltration galleries are made of wood, brick, or stone in the shape of a long gallery with small cross-section, with openings in the sides and bottom through which the water enters. A well-located gallery will intercept almost all of the ground flow in its locality, which is pumped from it direct or led from it by pipes or channel to a pump well.

An infiltration gallery may be considered as a large, horizontal well, and the statements above relative to ground flow apply to these as well as to wells.

At Newton, Mass., and some other cities wells are sunk along the line of the filter gallery and discharge into this, thus uniting the supplies. The same plan was adopted for the Brooklyn, N. Y., supply, the gallery consisting of several miles of large vitrified pipe.

An infiltration gallery is sometimes placed across the channel of a river which has a large underground flow, to intercept this. This method is particularly applicable to some of our western rivers, where the underground flow is at most times greater than the visible. The underflow of the Platte river was thus used foı the Denver water works. In many or most of such cribs considerable water is drawn from the visible supply, when there is any. The great probability that water so near the surface will be polluted has led to the abandoning of many infiltration galleries in the Eastern states; and they are recommended for such localities only as are beyond any sources of pollution.

In Parkersburg, W. Va., and a few smaller installations water is drawn from the river bed by placing a system of perforated pipes horizontally about 5 feet beneath the bed of a river and covering them with sand. The safety of this, as of any supply drawn from near the surface, depends largely upon the filtering effectiveness of the sand between the intaking pipe or gallery and any possible sources of pollution. In general 300 to 500 feet of fine sand or loam is reasonable protection against pathogenic bacteria. But the danger of cracks in soil or seams in rock must be guarded against; and water flowing between a surface of rock and any kind of soil fails to receive adequate filtration.

Quality. Springs are subject to contamination when from shallow surface strata or from deep ones which are very porous. A well-known illustration of the latter is the case of Lausen, Switzerland, where, in 1872, typhoid germs were found to pass through a hill and transfer an epidemic from one side of this to the other. The porous stratum here filtered out flour placed in the water, but was comparatively coarse, since the water passed through the hill in a few hours. A spring which shows little effect from droughts will generally be from a deep and extensive stratum, and free from all organic impurities.

Deep wells, and springs from deep and extensive strata, usually give a water containing little free oxygen and much mineral matter, the oxygen formerly present having united with the latter. The Grenelle well at Paris, 1780 feet deep.

contains no oxygen. The Ponce de Leon (Florida) well contains 319 parts per 100,000 of mineral matter, nine minerals being recognized, 195.8 parts being sodium chloride. "Old Faithful" geyser, Yellowstone Park, contains 139 parts of mineral matter, 63.9 being sodium chloride. Water from the two latter wells is not potable.

ART. 45. AMOUNT OF GROUND WATER AVAILABLE

Having determined the existence of an underground supply, its source, and the general conditions as to the geological strata, the next essential is to determine the quantity available.

The amount of water flowing in a given stratum may be represented by the formula $Q=aV$, a being the area of open spaces in a vertical section of the stratum across the line of flow, and V being obtained by test-wells, or by the formula on page 183. This is the maximum flow possible. But the total flow in a given stratum cannot *continuously* exceed the amount entering it by percolation from the catchment area; although it may do so temporarily by drawing upon the ground storage.

The percolation may vary within very wide limits, but will probably be about 60 to 70 per cent of the annual rainfall in sand, 25 per cent in sandstone, 15 per cent in limestone. English experimenters have found about 35 per cent to percolate through gravelly loam and chalk. The ground flow may receive water from not only its own outcrop, but also from that of any porous strata above or below it. In such a case all such interconnected porous strata may be considered together in figuring the catchment area. A well into one of these will draw to a greater or less extent from the others as well.

The total percolation into a stratum must generally fill all parts of such stratum which are below the hydraulic gradient. It is therefore possible that the volume per square foot of section flowing at any point may be either less or greater than the average percolation per square foot of vertical section at the outcrop; and this latter section may be but a small percentage of the actual exposed area, as at BC, Fig. 26.

Water-bearing soils not overlaid with impervious strata generally derive their water from the rainfall over the entire area of the basin or valley in which they are found, augmented by a considerable ground and surface flow from the adjacent hillsides, and by absorption of flood water by which they may be covered at intervals. In dry weather the flow is in some instances reversed, and water enters the ground from the river, lake, or ocean. This is generally true when the soil is of gravel or coarse clean sand, and when the river carries little silt. In any soils the river channel will probably be nearly impervious if the river at times carries much clay or loam in suspension.

The amount obtainable from such soil is therefore the rainfall upon it less that which runs off over the surface, the run-off from the area draining to it less that which flows across it as surface streams, and the amount which enters it by horizontal percolation from the water above normal stream height in time of flood and the vertical absorption of such flood waters as may cover it; less the loss by the several classes of evaporation. The amount derived from flood waters can generally be learned only by a study of ground-water heights in several wells at various distances from the stream before, during and after at least one or two floods. On Long Island it is estimated that 800,000 gallons per square mile per day can be obtained from sand where the only source is direct rainfall. Water in these surface soils flows into the nearest stream, ocean or other natural outlet until it falls to the level of such outlet. If drawn below this level by wells, a back-flow into the soil will usually take place. Salt water from the ocean has thus been drawn to a well, ruining it temporarily.

In the case of sandstone strata, the yield is more often a matter of the rate at which they will absorb and transmit water than of the amount that reaches the outcrop for absorption. This rate is extremely slow, generally only a fraction of a foot a day, and the amount that can be obtained from a given well or a given group of wells is limited by this rather than by the capacity of the stratum. The heavy draughts made upon the Potsdam sandstone by increasing numbers of wells have lowered

the hydraulic gradient at these wells, although it is not probable that the amount withdrawn is as great as that absorbed at the outcrop.

The exceeding slowness of flow through rock results in this flow's being almost constant through all seasons and years; the more extensive the stratum the greater being the uniformity. In sand and gravel the flow is more subject to variation from this cause, but here also the more extensive the stratum the less the variation. When the stratum feels the effects of droughts, the storage capacity of the soil may be called upon. If in Fig. 36, for instance, the ground water stand at the upper dotted line during the average season, it might during a dry one be gradually lowered to the lower line, the pumps having with-

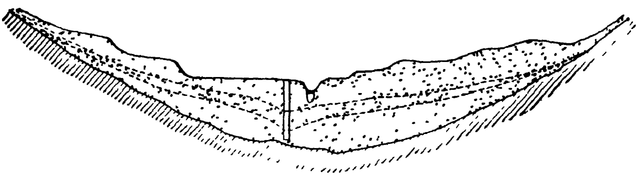


FIG. 36.—Illustration of Ground Storage.

drawn not only the contemporary seepage, but in addition an amount represented by the fall in water level. It is evident that the ability of a stratum to tide over a drought is measured by its area, depth, proportion of voids, and percentage of contained water which can be abstracted. The volume of voids will vary probably from 20 to 45 per cent of the total volume, depending little on the size of grain but much on the uniformity of size. The amount of the contained water which the soil will yield depends upon the capillarity and hence the fineness of grain. Thus, gravel will give up practically all and clay almost none of the contained water. Ordinary sand will surrender 60 to 70 per cent, and fairly permeable soils 50 to 60 per cent, of their water. An average sandy loam will yield about 20 per cent of its total volume. A stratum of 10 square miles area would yield about 418,000,000 gallons per vertical foot of saturated soil, i.e., soil below the ordinary ground-water level. If the daily

supply for six months was 10,000,000 gallons, while 12,000,000 was required, the additional might be obtained by lowering the ground water an average of 10 inches. The area, however, must be that of the ground-water surface and not of the ground; and as the water surface is lowered its area contracts, and hence the water level falls more quickly the longer the ground storage is called upon.

The amount of ground water considered in this article is the total flow; but it must be remembered that in almost no case is it possible to intercept all of this.

CHAPTER X

GRAVITY SYSTEMS

Art. 46. Definitions

A supply of water of the requisite quality and quantity having been decided upon, and the amount of storage necessary, if any, having been calculated, there remains the problem of conducting this to the consumer and distributing it in such quantities as shall be necessary, and under the necessary head.

The methods of bringing the water from its source to the point of utilization may be divided into two general classes —Gravity and Pumping Systems. In the former the elevation of the source above the point of utilization is so great that, if proper conduits be provided, the water will flow by gravity from the former to the latter and supply also the pressure head necessary. In pumping systems the source has not sufficient elevation to provide this flow and pressure, and the water must be raised or given sufficient pressure by some form of pump. The source may be higher than any point of utilization, and pumping still be necessary to overcome friction, to raise the water over an intervening ridge or other elevation, or to give increased pressure.

The several parts of a gravity system can be classified according to their functions under four heads: (1) The head works, or those by which the water is intercepted or obtained, treated to improve the quality if necessary, and introduced into the main conduit. (2) The main conduit, which carries the water from the source to the distribution system. (3) The distribution system, consisting of the various main and lateral pipes or other conduits, with their appurtenances, through which the water is distributed to the consumers. (4) Regulating works, such as reservoirs or standpipes, by which the head on the

distribution and the amount of water constantly available are determined and controlled. The last may be intimately connected, structurally, with either of the other three.

Art. 47. Head Works

The essential parts of the head works of any gravity system are a dam (unnecessary in a very few cases), forming a large impounding reservoir or one of smaller size down to a mere intake basin; and works for controlling the entrance of water into the main conduit. The amount of storage required has already been briefly discussed (Art. 37) and will be considered more in detail under the head of "Designing," Chapter XIV. This storage may be obtained in the impounding reservoir, or partly in this and partly in other storage reservoirs and distributing reservoirs. In general the aim should be to locate the storage where it will furnish abundant head on the distribution system, suffer a minimum loss by seepage and evaporation, be least subject to pollution, and be obtainable at the least cost. Other minor considerations also will require attention in some cases, such as danger of breaks in the main conduit owing to washouts at stream crossings, land slides, etc. These will be considered under the heads of "Main Conduit" and "Regulating Works."

When it is not necessary to store water in order to obtain a continuous supply, storage reservoirs may be omitted, and the impounding reservoir may be reduced to a slight damming up of the stream (which is, in most gravity supplies, a small one) to furnish sufficient depth and decreased velocity for the proper intaking of the water and to facilitate excluding sand and gravel from the conduit.

An impounding reservoir, which generally serves as a storage reservoir also, is located on the course of the stream or streams furnishing this supply; and in most cases is formed by placing a dam across a valley. If the valley above this point be long and narrow, the reservoir will be of this shape; and in many cases two or more valleys of contributing streams are united in one

reservoir; but if a natural basin can be found at a convenient elevation and location, this is to be preferred. It is desirable that the enclosing hills be steep, and that the valley be narrow where the dam is to be located, to avoid shallow water and expensive construction, but these conditions are not always obtainable. A basin or valley with little slope longitudinally will provide a given amount of storage with less height of dam than one with a steep channel, and is for this reason preferable. The geological formation should be such that there may be no loss of water by leakage under or around the dam or into another watershed.

The quantity of yield increases with the size of catchment area draining to the reservoir, and this increases as the dam is moved down-stream. But it is desirable that the reservoir be sufficiently high up the valley to enable the water to flow to the point of utilization, and also to furnish the desired head of water at this point. The distance from this point, and hence the cost of the conduit, it is desirable to make a minimum. Each of these conditions should be given due weight in choosing the location of the reservoir.

The first consideration should be maintaining the quality of the supply, the next the quantity. A sufficient head to avoid pumping should be aimed at; and this it may sometimes be desirable to obtain by constructing two or more reservoirs rather than by pumping from one at a lower elevation.

The location of the dam with reference to its stability is a matter of great importance. Sound bed-rock at or near the surface is desirable; or a thick bed of hardpan or clay, if an earth embankment is to be used. The distance away of materials for constructing the dam and convenience of transportation are important financial considerations.

In deciding upon the exact location of the dam, cost will generally be the controlling consideration. The elevation of the crest having been decided upon, that location is then best which requires the least expense for excavation and construction; and this is generally when the least quantity of material is required for construction. To decide this point, an accurate

contour-map should be made of the surface of the ground and of the rock or hardpan down to which the dam is to extend, and the approximate quantities required for several trial locations calculated; unless one location appears by inspection to be undoubtedly the best. In many cases a straight dam at the narrowest point is the best location; but conditions of topography are frequently met with which make more economical a dam whose center line is curved up-stream, or contains an angle. A detour may sometimes be desirable, also, to avoid a fault in the rock bottom; and a curve adds to the stability of a masonry dam.

Art. 48. Impounding and Storage Reservoirs

The relation between area and depth and the general shape of the reservoir must ordinarily depend upon the topography of the country; but the more regular the shore line the better, since small depressions in this are apt to cause stagnation of the water, and since in general shallow water and consequent danger from organic growths increase with the length of shore.

Shallow water not only encourages the growth of algæ and other vegetable organisms by admitting light and heat to the bottom and more polluted layers of water, but it also in summer causes the average temperature of the water to be higher. On the other hand, the deeper layers are apt to become stagnant in summer (see Art. 34) below a depth of 10 to 20 feet. The depth of non-stagnation can be increased by increasing the exposure to winds, as by clearing the shores of timber for some distance back from the reservoir; but this would also increase the evaporation (which is largely affected by wind) and the amount of sediment washed into the reservoir by storms, and is not to be recommended. With a given storage capacity, decrease in depth also means increased surface area and consequent loss by evaporation. A deep reservoir is hence advisable for all reasons except the formation of a stagnant layer, which may pollute the whole reservoir when the water "turns over"

in October or November. If the reservoir water contains little organic matter or unoxidized nitrogen, this objection is reduced to a minimum, and this condition of water should be obtained when possible.

It is difficult to prevent the growth of large amounts of plants and other vegetable organisms in water which is less than 4 or 5 feet deep; and it is therefore advisable that as little as possible of the reservoir water have less than this depth for any length of time. This requires that the shores should all be steep down to a depth of 5 or more feet below the ordinary water surface, and that all irregularities in the bottom which would cause shallows under any ordinary stages of water be removed. An ideal reservoir would be approximately oval in shape, with nearly vertical retaining-walls along the shores reaching a depth of 10 feet or more, the bottom rapidly reaching the maximum depth of 20 or more feet, at which depth it would have a uniform flat surface. Paved slopes are generally substituted for vertical walls to save expense; and the oval form can be only distantly approximated in practice.

The prevention of pollution of water by surface impurities has already been alluded to. It is equally necessary to prevent pollution of the water while in the reservoir. This pollution may come from the reservoir itself or reach the reservoir from the outside. The first is generally due to the improper cleaning of the reservoir before filling. Any organic matter that may be in the bottom of the reservoir decomposes slowly and the resulting nitrogen often supports vast quantities of algæ. Many reservoir sites have been cleared merely by cutting down the trees and bushes, leaving stumps, roots, grass, and other vegetable matter; but the majority if not all of such reservoirs for years afterward give trouble by the pollution of the water due to the decomposition of this matter. All stumps should be removed. Pockets of muck should be cleared out, or, if they are very deep, only the top 8 or 10 feet need be removed, and the holes should then be refilled with clean sand or gravel. As to stripping the soil from the reservoir site, see Art. 38.

All buildings should of course be removed from a reservoir

site, and all organic wastes deposited there by former residents. Privies in particular should be cleaned out and the soil for some distance around them be removed, the excavation being then disinfected and refilled with clean gravel and sand, or earth free from organic matter. Care should also be taken that the soil is not polluted by the workmen that construct the reservoir; to prevent which, closets should be provided below the dam site and the workmen compelled to use them.

It would be desirable to provide a concrete or similar artificial bottom for a storage reservoir, but the cost involved renders this impracticable; although a thin bituminous covering similar to a sheet asphalt pavement has been proposed for a large reservoir to reduce seepage.

Pollution from outside the reservoir may be from human beings defecating upon the banks or swimming in the waters; from organic matter deposited therein through malice or ignorance; and from leaves and organic dust blown into the water. (It is of course assumed that no stables, piggeries, or outhouses will permitted around the reservoir.) No picnics, bathing, or loitering around the banks of the reservoir should be permitted; to insure which a watchman should be constantly on hand. (It may be desirable, however, to permit driving around the reservoir.) To better permit watching the reservoir banks, and also to prevent leaves from falling into the water, it is well to clear all trees and other vegetation from a space 25 to 100 feet wide all around the reservoir; except that a hedge of evergreen trees or shrubs surrounding the reservoir just above high-water line, to prevent leaves blowing into it, is desirable.

The water entering the reservoir should bring as little matter in suspension as possible. For this reason, if the stream have considerable volume and velocity it may be desirable to provide a settling-basin at its entrance into the reservoir; generally by constructing a submerged weir across the reservoir from bank to bank near the mouth of the stream. Such basin must be cleaned out at invervals.

The conduit must receive water from the reservoir in such a way that there is no loss by leakage; that the mouth of the conduit can be tightly closed if desired; that no gravel, sand,

leaves, fish, ice, or other matter except water can enter it, and that the water can be drawn from different elevations above the reservoir bottom at pleasure.

It must also be possible to draw off and waste the water from the bottom of the reservoir when this is to be cleaned or repaired; or, as is sometimes desirable, to remove the bottom layers of stagnant water just before the "turn-over." These requirements are usually met by carrying pipes through the dam at the lowest point, and providing a gate house in which are valves by which the flow through these pipes is controlled, and screens to intercept suspended matters.

Spillways. If the reservoir should be approximately full at the time of a rainstorm, the run-off from this would need to overflow and be wasted. Provision for this is one of the most important details of reservoir designing, and insufficient allowance for it has caused more damage and loss of life than all other reservoir details combined. This waste water can of course be allowed to flow over the whole length of the dam creating the reservoir, but this is not permissible in the case of an earthen dam, and requires a most substantial construction along the whole foundation and front of a masonry or timber one. For these reasons the waste water is usually provided for by a spillway or waste weir.

A waste weir in the center or one end of a dam is frequently used, being practically but a part of the dam whose top is lower than that of the remainder and whose construction is more substantial. The waste water flows from this to the bed of the original stream. A spillway is frequently provided in the bed-rock at one end of the dam, a channel being cut in the rock at such elevation as to permit the water to overflow at the desired level. Where applicable, this method is generally preferable to a weir. A side spillway is practically a continuation of the dam along one side of the reservoir by a low wall whose top serves as the weir, the waste water flowing along a channel between this and the ground outside the reservoir. This construction is practicable only when rock is found near the spillway level along one side of the reservoir, to serve as foundation for the wall and bed for the waste-water channel.

In some instances the spillway is entirely separate and at some distance from the dam, being placed in a depression or " saddle " in the surrounding hills, to which the water is raised by the dam. A low masonry waste weir at this point will then suffice, and all danger from wash at the toe of the dam will be avoided, the water being discharged into another valley. This plan cannot often be adopted, but where practicable is an admirable one. In a few cases a tunnel in the mountain around one end of the dam has been provided to serve as a spillway and prevent wash at the toe of the dam.

The waste-weir or spillway should be constructed in the most substantial manner to withstand the shock of the overflow from the greatest floods. Its top must be designed to receive the blows from, and to pass over its crest, ice, logs, or any other matter brought down by the flood. It must so provide for the passing of all water, ice, and floating matter without any possibility of failure or of choking up, that the water in the reservoir can never under any condition reach the top of the dam.

The length of a spillway and the depth of water to flow over it demand careful consideration. The elevation of the top of the spillway, and not that of the dam, decides the elevation of the water surface in the reservoir and hence the amount of storage provided. This elevation then is the starting-point. The water should never reach such a level that its waves can rise above the top of the dam, which must therefore be higher than the spillway by an amount equal to the greatest depth of water on the spillway plus the greatest height of waves possible. If the dam be long, this additional height will add considerably to the expense; and to keep it at a minimum, the depth of water on the spillway must be decreased by increasing its length. But this may mean an increased cost due to the spillway, which is often much more expensive per lineal foot than the rest of the dam. The least expensive construction can ordinarily be ascertained only by comparing two or more plans. The depth of flow over the spillway during heavy floods should not be so shallow as to permit of ice, logs, etc., stranding there and forming an

obstruction. For the same reason piers, posts, or other obstructions which would be likely to catch ice, brush, or other floating matter should not be placed in the spillway.

The depth to be allowed for waves will vary with the length of the reservoir and consequent sweep of wind possible. Stevenson gives the formula

$$H = 1.5\sqrt{L} + (2.5 - \sqrt[4]{L}),$$

in which H is the maximum height of wave, in feet, and L is the length of the reservoir, in miles. If the reservoir be $\frac{1}{2}$ mile long, this formula would give $2\frac{3}{4}$ feet as the maximum height of waves; and if 2 miles long, $3\frac{1}{3}$ feet. Two feet is the least which should ever be assumed for wave height.

In calculating the capacity of a spillway—that is, the maximum rate of run-off from any storm—the method outlined in Art. 36 is recommended, rather than any of the formulas. The Melzingah (N. Y.) dam, which failed by overflowing in July, 1897, had a spillway which, calculating by any of the formulas, was sufficient for its drainage area of 1.1 square miles, but which proved its practical insufficiency.

It will generally be desirable to calculate the run-off from maximum rates of precipitation for 10, 20, 40 and 60 minutes on the drainage area, if this be small; and for 1, 2, 6, 12, and 24 hours if it be large. The run-off from a part of the area due to a short-period rate of rainfall *may* be greater than that from the whole area due to the lower rate of rainfall corresponding to the longer run-off time for such area. By using these few calculations to plot a curve representing the run-off due to maximum precipitation for different run-off times, the maximum rate of run-off may be determined. Given this rate, the depth of flow which it will cause over a spillway of assumed length and form may be calculated by use of proper hydraulic formulas.

ART. 49. DISTRIBUTING RESERVOIRS

If a storage reservoir be at a considerable distance from or above the point of utilization, a distributing reservoir is fre-

quently interposed in the conduit near such point, to relieve the distribution system of excessive pressure, lessen the liability of interruption of service, and permit the discharge of water at a high rate during short periods. The effecting of the first-named result by a lower distributing reservoir is apparent. The second result is usually obtained because, should there be a break in, or other interruption in the service of, the conduit between the impounding and distributing reservoirs, the latter would continue the supply during such time as would ordinarily be required to repair the conduit. A short conduit under pressure from a service reservoir will deliver at an unusually high rate with less loss of head than will a long one, since the total loss of head varies with the length; and, moreover, a size of conduit adapted to a given (temporary) high rate of discharge need be carried from the distributing reservoir only, which can be fed by the con-

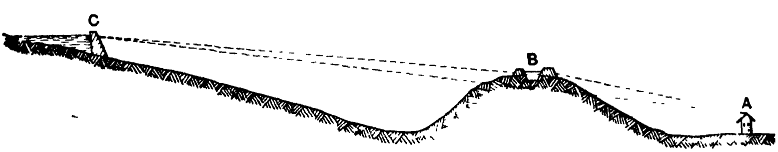


FIG. 37.—Distributing Reservoir.

tinuous flow through a much smaller one from the intake, thus saving the cost of making the latter line of larger size throughout. If, however, the utilization is continuous and constant in volume, the last reason for the use of a distributing reservoir is not applicable.

For example, in Fig. 37 a city, *A*, is to be supplied with water from a storage reservoir, *C*, there being a hill, *B*, but one-fifth the distance from *A* that *C* is, which hill is 200 feet above *A* and 25 feet below *C*. A supply for fire purposes or other heavy draught under a 100-foot pressure, to pass which would require a 20-inch pipe from *B* to *A*, would require a 27-inch pipe to bring it directly from *C* to *A*; while an average supply of one-fourth this amount, which would probably supply the ordinary consumption and keep *B* full from day to day, would require between *B* and *C* but a 16-inch pipe.

In addition to the above reasons, there is convenience and safety in having complete control of the supply to the distribution system located at a point near the city.

The damming of a valley, when one admitting of this is favorably located, is the least expensive method of forming a distributing reservoir. In most cases, however, such a reservoir is constructed on the top or side of a hill above and near the point of utilization. In such a situation part or all of the sides of the reservoir are in most cases partly or wholly in embankment. Stability and economy are generally best obtained by locating a reservoir on comparatively level ground, thus avoiding high embankments; and in no case should any part of the bottom of a reservoir be above the original ground surface. The location should generally be as nearly as possible on the direct conduit line from the storage reservoir to the point of utilization, and near the latter. Many distributing reservoirs have been located within the limits of the city they serve.

The reservoir capacity should be at least equal to the maximum consumption for two or three days. (A capacity equal to five days of maximum consumption plus a ten-hour fire flow is the standard of the National Board of Fire Underwriters.) It is desirable to have two reservoirs, or one divided by a partition-wall, that each may separately be emptied and cleaned without interrupting the service. This also permits the water to stand for two or more days and deposit any sediment which it may contain, the other reservoir being used meantime; but such use of a distributing reservoir is not generally desirable, unless possibly for one or two days a year of muddy water.

In plan the reservoir is frequently a quadrilateral, preferably with rounded corners. But this is determined by economical considerations, the greatest capacity being obtained at the least expense.

Since it is so much smaller, a distributing reservoir can generally be constructed more as theory dictates than can a storage reservoir. For example, the banks can all be given a steep slope and they and the bottom be paved throughout; the reservoir should be of a considerable depth, there being no

"turn-over" to avoid; it can be fenced in and all pollution from outside sources avoided; and is covered in some cases to insure this and to preserve a low temperature as well as to prevent the growth of algæ.

A distributing reservoir is fed by a conduit from the storage reservoir and discharges through one leading to the distribution system. It is provided with a waste-pipe to permit emptying it, as well as gates for controlling all these. It should be perfectly water-tight and stable, more particularly when in the midst of or near an inhabited section.

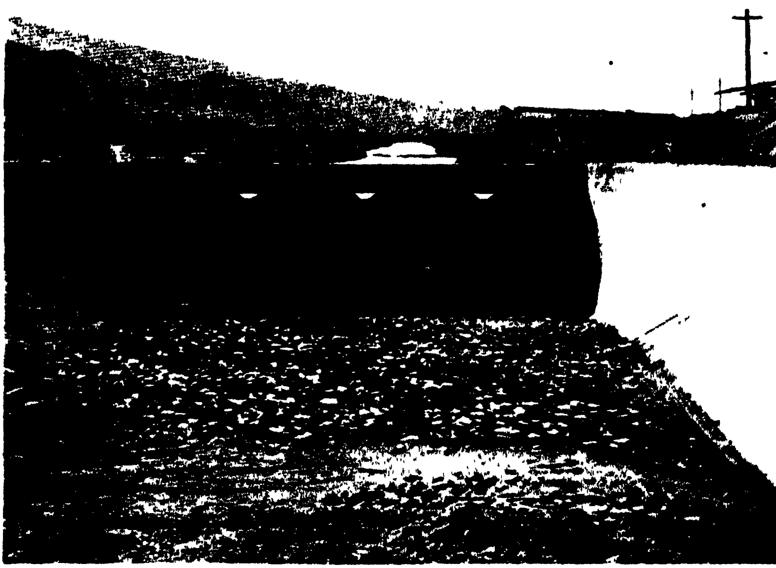


FIG. 38.—Intake Gate at Headworks of Los Angeles Aqueduct, California.

ART. 50. GRAVITY SUPPLIES FROM LARGE STREAMS

The above articles have considered the supply as being from surface water and small streams only; but in some cases a gravity supply is obtained from a stream of such size that no storage is necessary, a part only of the ordinary and flood discharges being diverted. A canal or flume leading from one bank

of the river will intercept a part of the flow; but it is also necessary that provision be made for intercepting a large part or all of it in time of low water, for excluding gravel and the heavier sediment during floods, and for drawing off at any desired rate at all times. This ordinarily requires a dam which will retain all the flow when necessary, but pass most of it during flood; and head-gates by which water can be taken into the conduit from the bed of the river during low water, but from near the surface during floods when the bottom flow is full of heavy sediment. There should also be provision for flushing out the deposit which will collect behind the head-gates and dam, for which purpose sluices at the bottom of the latter are desirable. It is also necessary to provide that the channel, particularly at low water, shall pass by the end of the conduit, and this may require spur-dams, and a sluice near this point.

It is particularly necessary in works thus situated that the foundations and all portions of every structure be of the greatest strength and solidity.

Art. 51. Open Conduits

A conduit between an impounding or storage reservoir and a distributing reservoir, and all conduits in irrigation systems, may be either open or closed and follow the hydraulic gradient, when they are called gravity conduits; or they may be closed and rise and fall with the surface, being under internal pressure due to their distance below the hydraulic gradient, when they are called pressure conduits. Conduits from distributing reservoirs, or from storage reservoirs on city supply systems where there are no distributing reservoirs, must, for the last part of their length at least, be under pressure.

The simplest form of open conduit is a canal excavated in the earth. To avoid loss by seepage, this is frequently lined with concrete or other material. Conduits are also constructed of timber, or of sheet iron or steel, supported on the ground or on trestles; or of masonry in or resting on the natural soil or on solid embankments. Gravity conduits are carried across valleys and streams by means of aqueducts, and through

mountains by tunnels. In some Western works, gravity conduits of concrete, stone, wood, and steel, aqueducts, tunnels, and pressure conduits are all found on the same line.

A canal must ordinarily follow quite closely the surface contour of the country traversed, having only such fall as will give the water the desired velocity. This may, in a mountainous country, lead to such detours as to enormously increase the length, cost and head lost. Where a reasonably straight course can be obtained, however, and the ground is fairly level laterally as well as longitudinally, a dug canal is generally the cheapest. If the general longitudinal grade is slightly steeper than that permissible for the canal, an occasional drop can be made in the latter, either by a vertical fall or as rapids, or the head can be consumed by gratings or contractions in the channel, wooden or masonry construction being used at these points.

The chief objection to canals is the great loss by percolation, which has been found in Utah to amount to 20 inches per day; and on the Erie Canal to from 35 to 100 cubic feet per minute per mile of canal 40 feet wide, or about 3 to 10 inches per day. The following table, compiled by Prof. L. G. Carpenter of the Colorado Agricultural Experiment Station, shows the daily loss by seepage on various canals.

	Feet.
Pleasant Valley and Lake Canal....	0.66 to 5.00
Greely No. 3, special case..........	30.00
North Poudre Lateral.............	0.6 to 1.00
Centreville and Kingsbury Canals...	6.00
Kings River and Fresno Canal......	0.6 to 1.70
Kern County Canals, sandy soil.....	1.00 to 2.00
Kern County Canals, sandy loam...	0.39 to 1.30
Erie Canal.......................	0.25 to 0.80

A long canal in earth may lose by seepage more water than it delivers. To remedy this, much can be done by admitting water heavily charged with clay in suspension and permitting it to pass slowly through the canal. In place of this, or if the

canal must be tight from the beginning, the sides may be puddled if materials for this are at hand. If they are not, or if still greater tightness is desired, a cement lining may be given the canal (such a lining has been applied with the " cement gun "), or a heavier lining of concrete or stone masonry may be used. Such a lining, by protecting the soil from the softening effects of water, permits the use of steeper slopes when desirable, thus economizing in lining.

Where an excavated canal is not adopted because of seepage, or of unfitness of the soil, or because the transverse slopes are so steep as to require a dangerous amount of embankment, but where the contour can be followed, one or both walls of the

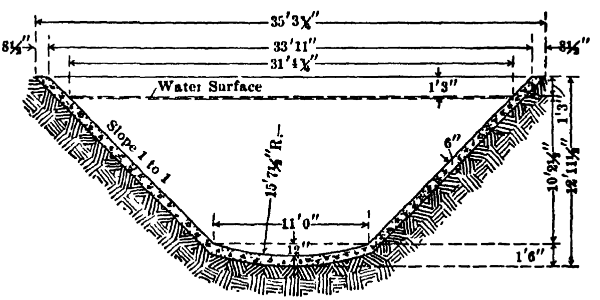


FIG. 39.—Open Canal, Lined with Concrete 6 inches Thick. Velocity full, 4 feet per second; Los Angeles Aqueduct.

canal are sometimes formed of masonry, the bottom of the canal being either bed-rock or concrete.

In place of a canal, an open conduit or flume of wood or steel is often used, resting upon the leveled ground where possible, but often upon trestles or embankments. These can be made practically tight, thus permitting no loss of water except that due to evaporation.

When the flume rests upon a level bench cut into a hillside it is called a bench-flume. A flume should never rest upon an embankment, which is sure to settle somewhat; and a bench-flume must be water-tight if resting upon earth, as any erosion of this caused by leakage would be fatal.

For crossing valleys at the hydraulic gradient, trestles or

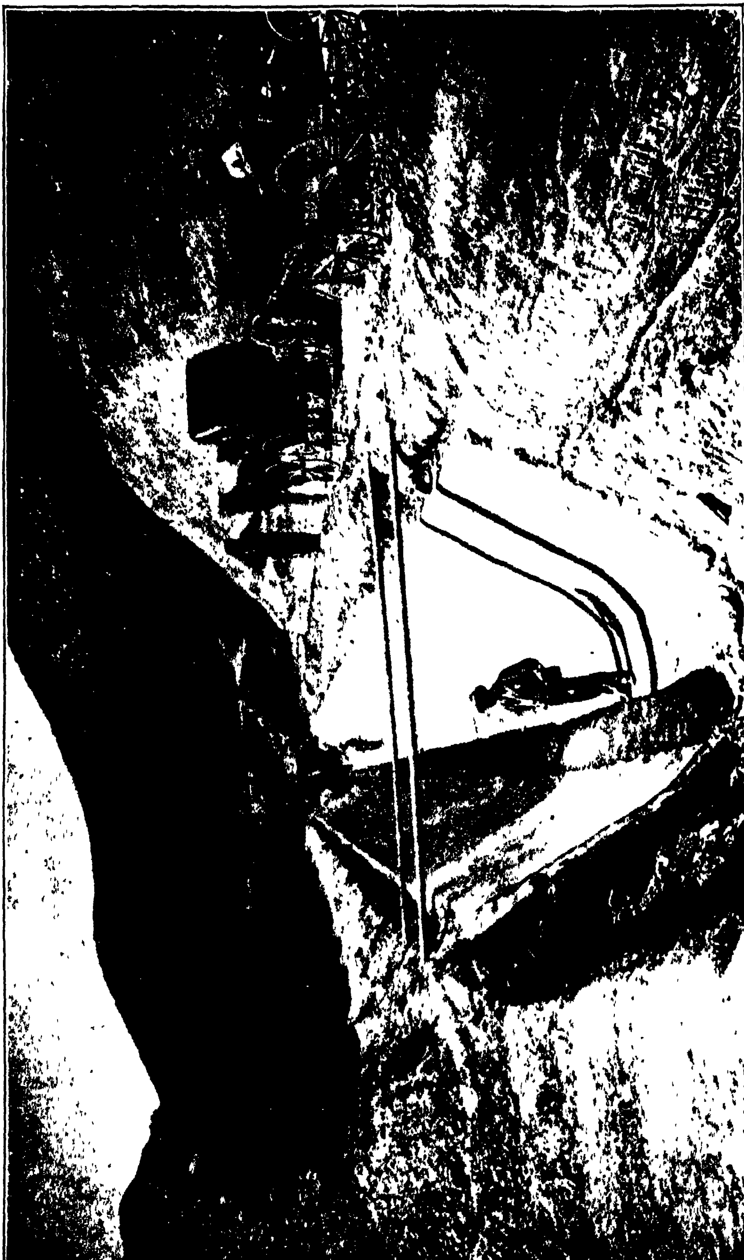


FIG. 49.—Open Canal Lined with Concrete Santa Ana Canal, California.

FIG. 42.—Conduit on Trestle.

aqueducts are most frequently used. The name aqueduct is generally given to a valley crossing of some magnitude and of substantial construction, as of masonry. High Bridge, New York, is an excellent example of this.

The longer water remains in an open conduit the greater the loss by evaporation, and by seepage if the banks are porous. For this reason, and to prevent the growth of weeds in earth channels, considerable velocity is desirable, reducing the cross-section dimensions; and is necessary also if the depositing of silt in the canal is to be prevented. But if the velocity become too great, the earth is eroded, or the metal, wood, or masonry abraded by the sand or gravel carried. Also, since velocity is obtained at the expense of head, a high velocity may cause too much fall in the canal and lowering of the pressure head at the point of utilization.

The growth of weeds may ordinarily be prevented by a velocity of 2 to 3 feet per second; and this velocity will also prevent the deposit of such matter in suspension as should properly be let into even an irrigation canal. Light or sandy soil is likely to be eroded by a velocity of 2 feet or more; in firm loam or clay a velocity of 3 feet is permissible; in brick-work, wood, or sheet-metal flumes a velocity of 5 or 6 feet may be allowed; but if the velocity exceed this a substantial construction of hard stone masonry should be provided, or steel or wood if no gritty matter is carried in suspension.

The grades which will give these velocities, in either an open or a closed conduit, depend upon the size, form of channel, and character of the wetted surface, and may be calculated by the proper hydraulic formulas.

The area of cross-section of the canal must be greater than the quotient obtained by dividing the maximum amount to be delivered per second, in cubic feet, by the velocity of flow in feet per second; the banks being at least 12 to 18 inches higher than the highest water surface. Also at any point the canal must have capacity for carrying the amount to be delivered at the lower end, plus the probable loss by seepage and evaporation between there and the point in question.

Recognition must be taken of the fact that consumption in cities is not uniform throughout the year, but may be 25 to 50 per cent greater than the average for one or even several days at a time in the summer, when the evaporation and seepage also are greater. The amount designed to be carried by the canal will be based upon these conditions, being at least 50 per cent greater than the average daily consumption, aside from the evaporation loss. Such variations as last for a few hours only will be provided for by the distributing reservoir if there is one.

In case more water were admitted to an open conduit than was being taken at the lower end, the banks or sides of the con-

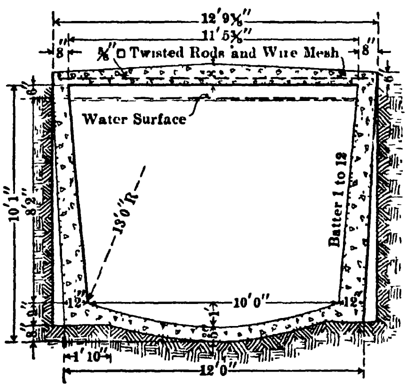


FIG. 43.—Covered Concrete Conduit in Excavation, Los Angeles Aqueduct. Velocity full, 4.6 feet per second.

duit would be overflowed, which would in most cases be disastrous. To prevent this, provision is made to remove the surplus water by specially constructed overflows, called waste-weirs or waste-ways, placed at intervals along the line. These must not only allow all water rising above a certain level to escape, but must provide a channel for carrying off the water without endangering the stability of the conduit.

In some cases no artificial conduit is constructed below the storage reservoir, but the discharge (regulated in quantity from time to time according to the demand) is conducted in the original channel of the stream to a distributing reservoir, another storage reservoir, or a conduit intake.

Art. 52. Closed Conduits

Closed conduits are provided to eliminate danger of pollution, loss by evaporation, and variation of temperature; but especially to permit carrying the conduit below the hydraulic gradient, where it would receive internal pressure and must be of a construction to resist this. Masonry conduits can do so to but a very limited extent, and are hence adapted to those loca-

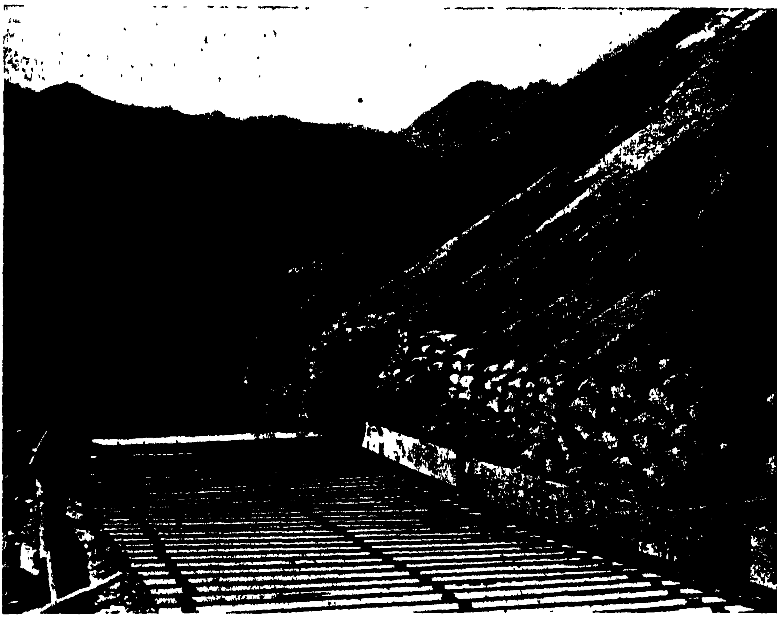


Fig. 44.—Covering Conduit with Concrete Roof to Prevent Slides Filling It, Los Angeles Water Works.

tions only which follow exactly along the hydraulic gradient. If such location requires considerable cutting, tunneling, embankments, trestles or masonry bridges, and other expensive work, metal or wooden conduits able to resist internal pressure are ordinarily employed, laid to follow the surface along a more direct route. Where the amount of water carried is very considerable, a trestle or even a masonry aqueduct may, however, be cheaper than a pressure conduit which rests upon or in the

surface at all points. A masonry conduit is in some cases made to resist internal pressure by constructing it as the lining of a tunnel in rock, the pressure being received and resisted by the rock.

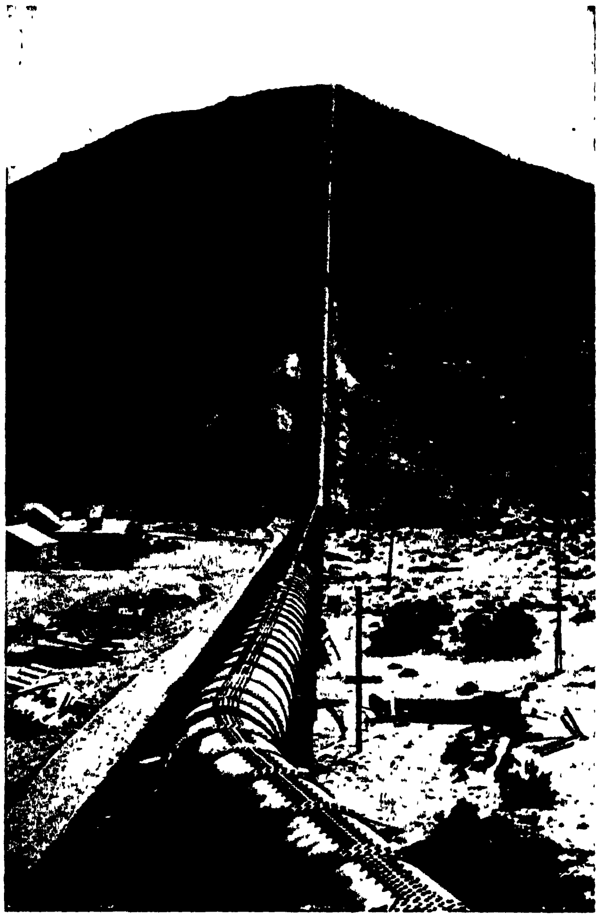


FIG. 45.—Steel Pressure Conduit, Jawbone Siphon, Los Angeles Aqueduct.

The majority of pressure conduits are constructed of steel plates or wood staves for the larger, and cast iron for the smaller sizes. Reinforced concrete has been used for heads up to 100 feet. The first pipes used in this country were of bored logs,

and spruce-log pipes eighty-five years old have been found in good condition.

When closed conduits are not under pressure, their size and

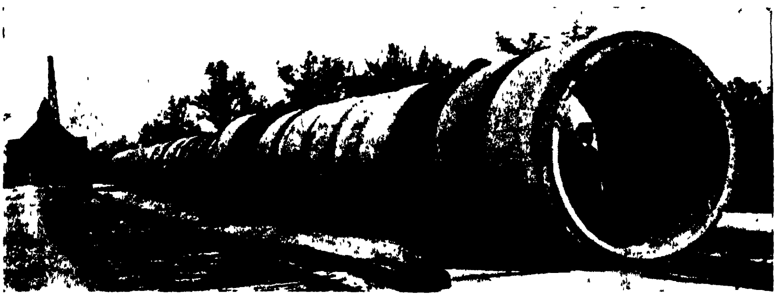


FIG. 46.—Reinforced Concrete Pipe for Pressure Conduit, Baltimore Water Works

grade are calculated as in the case of open ones; and overflows or waste weirs are similarly provided. When under

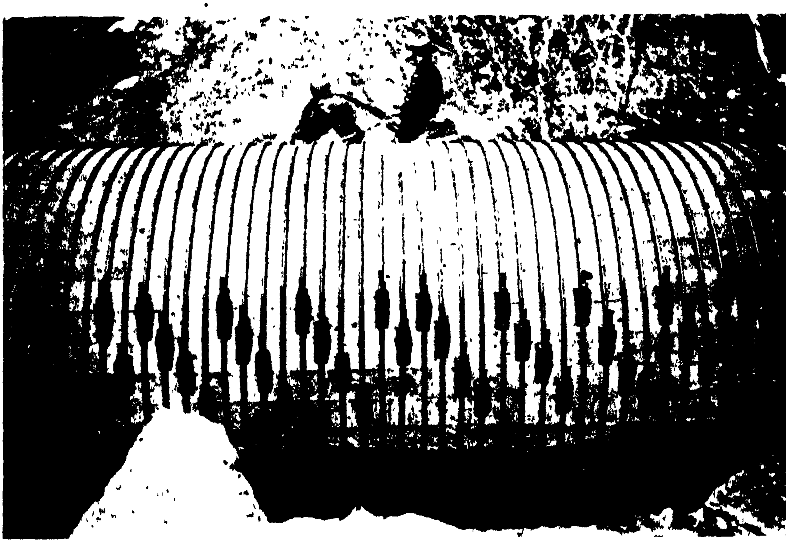


FIG. 47.—Side View of 74-inch Wood Stave Pipe.

pressure, they must always flow full, and the formulas for flow in pipes are applied. No waste weirs are then necessary except at the head-works; but flushing-out gates should be

provided at the low points for removing sand and other deposits, and air outlets and inlets at the high points.

The pressure to be resisted by the tensile strength of the walls of the conduit at any point is the hydostatic pressure due to the difference in elevation of the conduit at this point and of the water in the open conduit or reservoir at its head. This pressure is not attained when the water is flowing, but only when a gate at the lower end of the pressure conduit is closed. When flowing, the pressure head equals the vertical distance between the conduit and the hydraulic gradient, and this is the maximum pressure to be provided for if there be no gate at the lower end of the pressure conduit, or if an overflow be provided there at the level of the hydraulic gradient.

Both wood-stave and riveted-steel pipes have been constructed 72 inches in diameter and may be made yet larger. Rock tunnels can be made of any size.

Table No. 34

SOME CLOSED CONDUITS

Location.	Material.	Dimensions, Inches.	Length, Miles.	Velocity of Flow, Ft per Sec.	
Nashua, Boston	Masonry	126×138	7 0	6±	Gravity
Sudbury, Boston	Masonry	108× 92	15 9	3	Gravity
Croton, New York; Old	Masonry	89×101 5	..	2 218	Gravity
Croton, New York; New	Masonry	147 diam.	29 63	3 0	Pressure
Baltimore, Md	Masonry	144 diam.	7 0	..	Pressure
Denver, Colo	Wood	30 diam.	18 00	..	Pressure
Caldwell, Idaho	Wood	54 diam.	0 13	..	Pressure
Bear Valley, Cal	Wood	52 diam.	0.41	...	Pressure
Ogden, Utah	Wood	72 diam.	5 10	8 75	Pressure
Ogden, Utah	Steel plate	72 diam.	0 87	8 75	Pressure
Newark, N. J	Steel plate	48 diam.	21 00	5 0	Pressure
Rochester, N. Y	Steel plate	60 diam.	0 30		Pressure
	Cast iron	30 diam.	20	...	Pressure
Coolgardie, Australia	Steel plate	30 diam.	328 0	1 9	Pressure

The Catskill and Los Angeles aqueducts contain concrete-lined tunnels, gravity concrete aqueducts, steel pipes, inverted siphons and other structures of various sizes, too numerous to tabulate in detail, several of which have been illustrated in the previous pages.

ART. 53. LOCATION OF CONDUITS

The location of a conduit when the loss of a few feet of head is not important is largely a matter of economy. When every available foot of head must be preserved, however, the alignment must be approximately straight and the grade as light as will give the desired velocity. To avoid excessive evaporation at low velocities, and to permit following the irregularities of surface in an air-line location, closed conduits, in many or most places under pressure, are used. There may be locations where the loss of head in passing around a basin or pocket will be less than in going directly across it with a deep loop or inverted siphon. A true siphon is to be avoided where possible, although a number of these are in use.

If the question is one of cost only, a more circuitous line is frequently preferable. Although longer, it may avoid deep cuts and tunnels, may be of gravity section for most of its length, may have considerable fall and hence smaller cross-section, and deep inverted siphons, calling for strong, high-pressure conduits, may be avoided. In many instances, however, deep cuts, tunnels, or inverted siphons may be the less expensive. Estimates of cost for different routes must generally be made for each section to determine the most desirable location.

Sharp curves should be avoided, since they cause loss of head and erosion of the outer banks of canals. In general the minimum radius of curvature of the inner side of a conduit should be at least two or three times the product of the depth of water and the velocity of flow in feet per second.

Streams should always be crossed at a safe distance beneath their beds or above their highest flood-lines.

Swamps and other soils affording poor foundations should be avoided if possible; if not, artificial foundations must be carried down to rock or firm soil.

ART. 54. DISTRIBUTION SYSTEMS

The distributing system of a city supply is always composed of pressure pipes. The pressure varies, in this country, between 10 and 200 pounds, with perhaps exceptional cases

greater than this. The great majority lie between 50 and 150 pounds. The static pressure in any part of a system fed by reservoir is that due to the difference in elevation of such point and the level of water in the distributing reservoir. The pressure, in pounds per square inch, equals 0.434 times this difference of elevation, in feet. All portions of a distribution system must be designed to withstand the static pressure, plus pressure due to water hammer. (See Art. 80).

The two chief objects of a city water-supply system are: to furnish water for domestic, manufacturing, and similar purposes, and to afford fire protection. Use is also made of it in sprinkling streets and lawns, flushing sewers, etc. For these purposes the distribution system should reach every building, and permit of placing fire hydrants at distances apart of not more than 500 feet in all occupied streets. The filling of sprinkling-carts, supplying of fountains, and flushing of sewers must also be provided for. The standard of the National Board of Fire Underwriters for distribution of fire hydrants, in communities of different sizes, is as follows:

Fire Flow Required, Gallons per Minute	Average Area per Hydrant, Square Feet
WHERE FIRE ENGINES ARE USED	
1,000	120,000
2,000	110.000
3,000	100,000
4,000	90,000
5,000	85,000
6,000	80,000
7,000	70,000
8,000	60,000
9,000	55,000
10,000	48,000
11,000	43,000
12,000	40,000
DIRECT HYDRANT STREAMS	
1,000	100,000
1,500	90,000
2,000	85,000
2,500	78,000
3,000	70,000
4,000	55,000
5,000 and over	40,000

Not only must the pipes of a system reach all these points, but they should be of such size that the maximum desired rate of flow may be obtained at any point. It is desirable that the pressure be such at every fire hydrant that at least two streams through 400 or 500 feet of fire hose may be thrown to the top of the highest building; but this is not always possible, especially where the buildings are excessively high.

An additional requirement is that the pipes and all attached appurtenances, such as fire hydrants and valves, be amply capable of withstanding the greatest pressure to be brought upon them.

It is desirable that all parts of the system be durable, requiring to be renewed only at long intervals of time. Also that it be so arranged that repairs, alterations, extensions, or renewals may be made, or breaks occur, at any point without interfering with the service at any other point. All parts of the system should be easily located and accessible.

It is especially desirable that the quality of the water be not impaired by any part or condition of the system; and that there be no waste or leakage at unknown or inaccessible points.

The reaching of all buildings usually requires that a pipe pass along one side of every lot. This does not always necessitate laying pipes in each street, since in sections of many cities all buildings face upon one set of parallel streets, while those crossing these pass but the sides of corner lots; in which case, if there are pipes upon the main streets only, they will pass all houses. In such sections of a city the lengths of cross streets included between the main streets seldom exceed 300 to 400 feet, and if fire hydrants be placed at the corners no intermediate ones on the cross streets, and hence no pipes, will be necessary. The filling of sprinkling-carts and providing for sewer flushing and fountains will seldom require any pipes not demanded by the above considerations.

The arranging of the system to provide for restricting the interruption of service consequent upon repairs and breaks demands that a means be provided for cutting off any small

section of it from the rest of the system. The smaller the section cut off the less the inconvenience caused. It is desirable that such cutting off may be effected quickly in case of a break

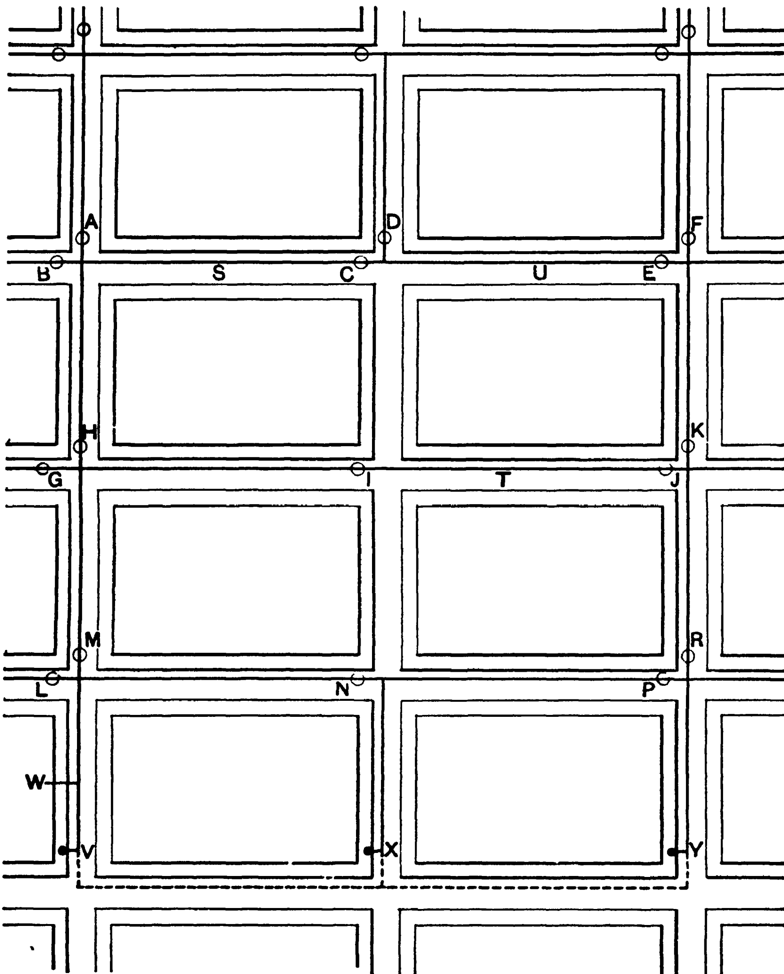


FIG. 48.—Location of Mains and Valves.

or other accident. This is generally attained by inserting gate valves in the lines of the pipes. These can be placed at any desired distance apart, but one on each line at each corner, or at intervals of 500 to 750 feet apart where the blocks are longer

than this, is probably as close as it is generally desirable to place them. (The standard of the National Board of Fire Underwriters is such spacing of gates that no accident will require cutting out of service more than 500 feet of main in a high value district, or 800 feet in other sections.) When so located, not more than four gates need to be closed to cut out any short section less than one block long. Thus, in Fig. 48, a break at *S* would require the closing of gates at *A*, *B*, *C*, and *H* to exclude all water from that point while making repairs. The point *T* can be cut out by closing *I* and *J*; and *U*, by closing *C*, *D*, and *E*. In neither of these cases will any section except that between the gates named be deprived of water. If gates be placed on only every other corner on each line, twice the length of pipe will be put out of service by breaks or repairs.

Of the pipes and conduits which are usually placed in a street, all cannot occupy the center. For several reasons it is generally desirable that the sewers occupy this position; and the water pipe must therefore be on one side of the center. Which side is not a matter of very great importance; but the north side offers the advantage of being warmer, and hence giving less opportunity for the freezing of the pipes. The same side of all streets throughout the city should be used for the water pipe, however—as the north and west sides—to facilitate ready location.

In order that the pipe may be quickly and readily located, it should be placed at a uniform distance from some fixed line of the street in all parts of the system. This line may be the center of the street, the curb, or the property line. The use of the centre line requires that this be first located, thus more than doubling the time and labor required. When sidewalks are of different widths and curbs are not constructed in all streets, the use of a constant distance throughout the city from either curb or property line is impossible or inadvisable. The author has adopted, as a satisfactory rule, a distance from the property line of such multiple of 5 feet as will bring the pipe between 5 and 10 feet from the curb. The property line, usually indicated by a fence or house front, is easily found,

and there is little danger of using the wrong multiple of five for the distance. For instance, in a 60-foot street with 12-foot sidewalks the pipe would be located 20 feet from the property line.

The hasty location of a gate is often most important, as in case of a break, when the flow in that section must be stopped immediately to prevent further damage. In many systems the memory of the superintendent or the maps in the office must be consulted before a gate can be located, and if either or both of these be temporarily lost great delay and damage may result. This can easily be avoided by placing the gates systematically, as in line with the curbs, in the middle of the cross-walk, or in range with the property line. The last named offers the most ready method of location when the curb and cross-walk are hidden under the snow. The gates should be on a uniform side of the corner, also. In Fig. 48, all pipes being upon the north and west sides of the streets, and all gates on the northwest corners and in range with and at a known distance from the property or fence lines, their exact location can be found in a few seconds.

Post fire hydrants (that is, those which stand above the ground 2 to 4 feet) can readily be found if their approximate location is known. (In winter, snow should never be allowed to cover them, but they should be kept shoveled out and ready for instant use.) Flush hydrants, which are flush with the street pavement, should be located uniformly in a manner similar to that used for gates. Any other underground fixtures will be so few in number that the location of each can generally be fixed in the memory as well as recorded in the office.

The maintenance of the quality of the water requires that all portions of the pipe, gates, and other appurtenances which come into contact with the water shall be such as to give to it no taste or odor or objectionable soluble matter. Some coatings used on iron pipes and gates give a tarry taste to the water for a short time, but this is not injurious and soon passes away. The most frequent violation of this principle is in the use of lead or lead-lined pipes, this metal being soluble in some waters. In general, soft waters dissolve lead and temporarily hard

waters do not. The action upon a given lead surface diminishes with time, but may be so considerable as to cause lead poisoning in consumers. The only sure method of determining whether a given water attacks lead is to test it by actual contact with a bright lead surface for several days. Zinc also is soluble in some waters, but is not so dangerous to the human system as is lead. For waters which attack these metals, service pipes of iron lined with tin or cement are recommended.

If the water contain any organic matter, its stagnation in a pipe deprived of light and air is likely to cause offensive decomposition, although this matter might have been of a harmless nature. Stagnation also favors the deposit in the pipes of any mineral matters carried in suspension. It is therefore desirable that the water in all parts of a distribution system be kept in motion as much as possible. If but one house be located in the southwest block of Fig. 48, as at *W*, the water between it and the dead-end, *V*, will lie stangant, and if it become putrid may contaminate the water entering *W*. Or if no water be used at *W* for some time, the first to be drawn may be unfit for use. Another objection to the dead-end is that water drawn from the fire-hydrant *V* must all pass through the pipe *VM*; but if pipe be laid along the dotted lines, connecting with the system at *V*, *X*, and *Y*, water will reach the hydrant from both directions, and the velocity of flow and friction-head in *MV* will be reduced and the discharge from the hydrant increased. For both these reasons dead-ends are to be avoided where possible. When they cannot be, a fire hydrant or other method of flushing out the "dead" water should be provided at each dead-end, and opened at intervals throughout the year.

It is evident that, with a system of pipes in which the water may circulate freely in all directions, the proper determination of sizes is a matter of some difficulty, since water will reach any point not in a dead-end through a number of pipes, and not through one alone. This determination will be considered in Art. 80, and the material and strength of pipes in the same article.

If there be considerable difference of elevation in different sections of a city, a desirable pressure head in the upper ones will occasion an excessive head in the lower, if they be connected. This difficulty is overcome by dividing the city into two or more districts, each with its own distribution system, and generally its own distributing reservoir. In a few cases the "low service" system receives its supply from the "high service," but a pressure regulator is placed at the junction, which cuts off the supply when the pressure in the lower system exceeds a certain amount.

Some cities have constructed in the low-service district a supplementary fire system of mains of extra strength, which are connected with the high service, and thus obtain high pressure for fire service in the lower and business part of the city. Still more of the larger cities have constructed in their business districts special systems of mains of extra strength for fire service, the extra pressure being furnished by special pumping plants.

CHAPTER XI

PUMPING SYSTEMS

Art. 55. Where Required

If the impounding reservoir, stream, or lake forming the source of supply is not at sufficient elevation to give the required pressure in the distribution system, in addition to overcoming the friction in the conduit, pumping must be resorted to. If the elevation affords sufficient pressure for the low-lying part of the city, but not for higher points, then the supply for the latter only must be pumped. Ground waters must be pumped in the great majority of cases, but not all. As a general statement, subject to many exceptions, however, impounded surface supplies do not need pumping; while river, lake, and ground waters do.

Since a pump, like any mechanism, is subject to interruption or discontinuance due to breaks or other causes, while gravity never ceases to act, a gravity supply is preferable to a pumped one, other conditions being equally favorable. Also the first cost of a gravity supply is the final one so far as moving the water from source to city is concerned, while pumping involves a continual expense for power, pump upkeep, and repairs, and pumping station employees.

If a ridge higher than the hydraulic gradient of a gravity conduit lies in the line of this, it will generally be better to tunnel this at or below the gradient than to pump over the ridge. If such a tunnel cost less than a pumping plant plus the capitalization of all expenses of pumping and renewal, it offers the more economical solution of the problem, as well as the more reliable.

Art. 56. General Design

The water which is raised by a pumping plant must pass through a pressure pipe, or pumping main, either directly to

the point of utilization, or to a storage or distributing reservoir or standpipe. The first is called the direct-pumping system. This requires, aside from the distribution system, the construction necessary to conduct the water to the pump; the pumping plant and building in which it is housed; and the pumping main. The pressure in a city supply, with direct pumping, depends upon that in the pump cylinders, and when the pumps cease working and water is drawn by consumers the pressure falls, approaching zero. If the pump in such a plant breaks down, the suppply is immediately cut off, unless a duplicate plant be provided. A duplicate plant is always desirable in every pumping system, but is imperative in a direct-pumping one.

In pumping to a reservoir, or an indirect-pumping system, there is required a reservoir, a pumping main to it, and a conduit from it to the distribution system. In some plants the pumping main passes through and forms a part of the distribution system, and also acts as the conduit between this and the reservoir, the water passing directly to the distribution system, but being augmented by that from the reservoir when the amount pumped is less than the consumption, and the surplus, when it is greater than the consumption, passing to the reservoir. Such a system is called a direct-indirect.

In the direct-pumping system the pressure in a city supply is made to vary constantly by variations in the rate of consumption which the rate of pumping cannot be made to follow exactly. Moreover when, as in the case of fire, an unusually heavy draught is suddenly made, the pumps may not meet it immediately and the pressure will fall (and may even become negative, if pumping fire engines are used) until the pump is speeded up or another one thrown into service. There is, however, the advantage that except during fires the pressure can be kept at the least which is satisfactory for ordinary service, thus reducing pumping cost, loss from leakage, and stress on house plumbing. But the raising of pressure (say from 40 to 75 pounds) during a fire often causes breaks due to deterioration not noticed under low pressure, and this and the possibility

that boiler trouble or a sleeping engineer may seriously delay the increased pumping for a fire are serious objections to direct pumping.

When, for want of an elevated site or other reason, a reservoir cannot be had, a water column is frequently introduced to equalize the pressure; and if it be given considerable size of cross-section it will serve temporarily to supply a deficiency in pump delivery. Such a water column generally takes the form of a standpipe or water tank. Where a water cushion or pressure regulator alone is desired, a tall pipe of small section is used; as at Wichita, Kan., where a pipe $2\frac{1}{2}$ feet in diameter and 150 feet high was erected. Where storage also is desired the cross-section area is considerably increased, as at Houston, Tex., which erected a standpipe 30 feet in diameter and 150 feet high, holding 144 times as much water as the former. The standpipe may be placed at or near the pump, or at some point in the distribution system, the highest ground in or near the city being ordinarily chosen.

Where there are two or more high areas to be served, a standpipe may be placed at the highest part of each, and these be kept filled by the indirect pumping method, while a reservoir at the same or at a lower elevation serves the lower part of the city. In some cases a small pumping plant is located at a reservoir which is filled by either pumping or gravity, drawing water from this and pumping it into a standpipe or a higher reservoir to serve a high-level district. In a few cases all the water is pumped to a high-level reservoir, and high-level and low-level distributing systems are connected by a pipe in which is set a pressure regulator, which prevents too high a pressure on the low-level. Or both high-level and low-level systems are connected to the same pumping main, a pressure regulator being placed on the connection to the latter. Such a regulator will hold the pressure constant with a variation of not more than a pound or two. A high-level reservoir will serve both levels in such case. In one city two reservoirs on opposite sides of a city and at different elevations are fed by direct-indirect pumping from the same plant, flow to the lower being cut off

automatically when it is full; and at each of these reservoirs is a pumping plant serving an area higher than the reservoir, a standpipe being placed in each.

These are a few of the combinations that can be made by use of pumping plants, pressure regulators, reservoirs, and standpipes. Different local conditions may often be met advantageously by others.

Where there is a purification plant connected with a pumping system, it is generally necessary to lift the water from river or lake to the purification plant through a vertical height of 10 to 25 feet, special low-lift pumps being provided for this purpose, while high-lift pumps supply the necessary pressure to the purified water. Both sets of pumps are placed in the same building.

Where the supply is drawn from wells, also, two sets of pumps are generally necessary, one to lift the water to the surface, the other to do the service pumping. The reason for not doing the entire lifting with one pump is that no pump that can be used in a well has yet been devised which can develop anything like the efficiency of the pumps used for ordinary surface pumping against a high head. Also there are ordinarily several wells, and the relatively small capacity of the individual pumps is also an obstacle to high efficiency. Where, however, the entire supply can be obtained from one well in which the water rises nearly to the surface, a high-lift, high-duty pump may be placed in a pump pit sunk as much as 50 feet below the surface, thus avoiding double pumping. There are a few localities where the artesian head is so great that the full capacity of the well (or at least a sufficient flow therefrom) can be obtained at the ground surface without well pumping; but ordinarily the flow can be so greatly increased by lowering the level in the well that this is more economical than to increase the number of wells. In some cases the ground-water level is 100 or more feet below the ground surface, and then well pumping is of course imperative. (Raising water by the air lift is here included under the head of pumping.)

Water can be raised by suction about 34 feet theoretically,

but in practice the suction lift should not exceed 22 to 25 feet; and if centrifugal pumps are used, it should be kept less than this. In general, the less the suction lift the better. If air leaks into a suction line, energy is lost and pump troubles are caused; and it is difficult to make a long suction line perfectly air tight. For this reason the pumps should be as near the source of supply as possible, unless the water can flow from the source to them by gravity. For both of these reasons, the location of the pumping plant is more or less closely determined by that of the supply.

In the case of indirect pumping, the nearer the reservoir is to the pump the shorter the pumping main and the less the friction loss, and consequently the greater the economy and the less the probability of a break in the all-important pumping main. The same is true of direct-indirect pumping also, but to a less degree; for in this a large part of the distribution system assists in carrying the water from pump to reservoir, thus reducing friction loss and the effect of a break in any one line. In such a system also there is an advantage in having the reservoir on the opposite side of the city from the pump; for then, in case of a break in the main feeder from either pump or reservoir, the other will supply the distribution system; also, a fire far from the pump would probably be nearer the reservoir and could be supplied from the latter with much less loss of head by friction than if all the water must be supplied by the pump.

The capacity of the pumping plant must be such as to assure under any possible conditions that the amount of water necessary for maximum consumption can be supplied at the desired pressure at any time, or at most on a few minutes' notice. (The consumption and pressure necessary have previously been discussed.) This requires provision against the breaking down of any one pump, allowance for the difference between service and test efficiencies, for the increasing friction in mains with age, variations in height of water at the source of supply or in the pump well, and all other exigencies of actual service. The pressure required at the pumping plant to produce the desired head at any given part of the distribution system is that necessary to raise the

water to the hydraulic gradient calculated by adding to such head the friction in the various mains traversed by the water in passing from the pump to such point.

Art. 57. Intakes

If water is drawn from a reservoir or other deep and quiet body of water, the suction from the pumps can pass directly into this; or the water can be led by gravity through a pipe or open channel to a pump well. A pump well is always preferable to a long suction. If the supply is from a lake, the same may hold true; but in some cases it is desirable to run the suction pipe, or gravity intake pipe to the pump well, out to a channel or other point a considerable distance from shore to avoid local pollution or to reach deep water. Some intakes in the Great Lakes are carried 4 or 5 miles from shore. In the case of a lake which does not receive pollution from along its banks, the purer water is almost always near the outlet rather than the inlet, on the windward rather than the leeward side, and in the channel rather than in bays or other stagnant water. If sewage or other foul matter enters the lake, the intake should be located so that prevailing winds and currents travel from intake to sewer outlet rather than vice versa, and on the opposite side of the main current or channel from such outlet if possible. In the case of these, and in fact of any intake in deep water, it is desirable to provide at the end of the intake pipe a tower or crib with ports at different elevations so that water can be drawn from that depth which, at any given time, will be freest from impurities, silt, ice or other objectionable matters. In the case of large lakes or of deep or swift-flowing rivers, the intake tower may need to be of heavy masonry on solid foundation to resist storms, ice floes, floating logs, etc.

Where water is drawn from a river, care should be taken to locate the intake at that point which will give the purest supply at all times. It should not be placed in an eddy, or in shallow or slack water at one side of the river, since matter in suspension is apt to collect here. It should always be at

a distance below the surface at least twice as great as the diameter of the intake opening. It should be at such an elevation above the bottom that no sand or silt carried by the bottom layer of water can enter it. It should be placed below rather than above any pond or slack-water, unless additional pollution is added to the water there. It should generally be placed above the city, to avoid the pollution by street drainage which probably enters the river, even if no sewers discharge into it. A screen is generally placed over the intake to exclude fish and floating matter; and it should be contained in and protected by a crib or masonry structure.

If the river (or smaller stream) has not sufficient depth to permit the submergence described, combined with elevation above the bottom to prevent entrance of heavy silt, it may be necessary to place a dam across the stream to create the required depth of water. In such case it is desirable to place a blow-off through the dam at the level of the old bed of the stream and near the intake opening, to be used from time to time to scour out deposits of sediment that are apt to collect behind the dam.

Where water is drawn from river or lake in northern latitudes, anchor- or slush-ice collecting and freezing around intake openings sometimes gives a great deal of trouble. This is caused by needles of ice which are prevented from freezing together by the motion of the water, and, drawn into the intake in this condition, freeze upon and choke the screen bars. This seldom occurs in still water or on the windward side of a lake or river; and is less likely to occur if the intake opening is some distance below the surface, or if it is of such an area that the velocity of flow into it is less than $\frac{1}{2}$ foot per second.

The intake pipes should be well buried in the bed of the river or lake, to prevent injury by currents or waves, by the anchors of boats, or from any other cause. They are sometimes made of wrought-iron or steel pipe, but cast iron is more durable. The intakes for several large cities are tunnels in rock carried out under the lake or river bottom, the tunnel terminating in a tower or crib.

If the supply is from tube wells and the water level is lowered in these by pumping, there is generally a pump in each well which delivers the water into a collecting main, which in turn leads it to a small basin or suction well. If, however, it is not desired to draw the water level down more than 20 or 25 feet below the level of the pump, the pump suction may be extended to connect with each of the walls but so connected that any well or wells can be cut off from the suction by closing a valve. This plan offers the serious objection that a long suction with the various connections and valves can with difficulty be made and kept tight, and air entering gives pump trouble, which even an air drum and air pump to collect and remove the air from the suction line cannot entirely prevent. Also it is difficult to regulate the flow of water from the individual wells when some admit water through the strainer more freely than others, or when they tap different water-bearing strata with various artesian heads. Where, however, the entire supply can be obtained from wells, none of which need be more than 100 feet or so from the pump, the direct-connected suction plan has been employed in several cases.

The motive powers adopted for raising water include man or animal power; water-wheels and other hydraulic machinery; windmills; gas, gasoline, oil, and hot-air engines; steam engines; electric motors; compressed-air engines; and in fact all kinds of motive power are used for driving pumps. Compressed air and steam are also used directly without the medium of pumps, in the air-lift and steam siphon.

The pump and motor are generally placed in the same building, and in most cases are really parts of the same machine. Animal power is not used in this country for any but private supplies. Windmills are used on few public supplies, not being adapted to pumping large quantities of water, and depending upon uncertain winds for power. Gas, gasoline, oil, and hot-air engines give excellent service in many small plants; and the three former in plants of considerable size. Electric motors are in quite common use for small plants, and for a few large ones. Hydraulic machinery has been used in several

plants, generally where the stream from which the water was taken could furnish the power also. Since streams furnishing hydraulic power are liable to periods of low water, a supplementary steam or internal-combustion engine plant is generally necessary to insure non-interruption of service.

By far the greatest number of pumping plants have steam as a motive power, this being adapted to the largest as well as the smallest engines, and being under control at all times; although an immediate increase in delivery in case of fire cannot be obtained, in which respect internal-combustion or electric motors are preferable.

TABLE NO. 35

METHOD OF SUPPLY TO CITIES AND TOWNS IN THE UNITED STATES

Method of Supply.	Districts.				Total.
	North-eastern.	South-eastern.	North-Central.	West-ern.	
Gravity	490	41	11	194	736
Gravity and Pumping:					
Direct	62	7	2	15	86
To reservoir	38	1	2	13	54
To stand-pipe	11	2	2	5	20
Direct and to reservoir	1	0	0	1	2
Direct and to stand-pipe	3	1	3	1	8
To reservoir and stand-pipe	4	0	0	8	12
Total	119	11	9	43	182
Pumping:					
Direct	74	33	221	90	418
To reservoir	128	62	79	114	383
To stand-pipe	245	139	218	358	960
To reservoir and stand-pipe	26	11	14	20	71
Direct and to reservoir	33	11	27	45	116
Direct and to stand-pipe	41	20	115	185	361
Direct and to reservoir and stand-pipe	3	2	18	10	33
Total	550	278	692	822	2342
Natural pressure	0	9	2	10	21
Grand total	1159	339	714	1069	3281

In deciding upon the motive power, consideration should be had of the accessibility of the plant and its general location. An electric plant requires that power be obtainable from an electric power station where power can be generated more cheaply than at the pumping station. Hydraulic machinery requires a constantly abundant supply of water power. Steam plants require coal, wood, oil, or gas to supply the heat, and the means of bringing these to the plant must be considered. It may be more economical to place a steam plant using coal near a railroad and some distance from the source of water supply, and bring the water by gravity to the pumps, even though this require a somewhat greater lift and cost of construction; or to locate a power plant as indicated and carry power in the form of electric current to a motor-operated pumping plant.

CHAPTER XII

PUMPING AND PUMPING ENGINES

ART. 58. PUMPS

ALL pumps in common use in water-supply plants may be divided into two classes, those that exert pressure directly upon the water through the medium of a piston or plunger, and those that create pressure in the water by centrifugal force. The former are reciprocating pumps, the essential feature of which is a piston or plunger moving back and forth over a straight path; the latter are centrifugal pumps, the moving part consisting of revolving vanes that give to the water a rotary motion. There are also the air lift, which is not strictly a pump; and pneumatic ejecters, diaphragm pumps and other devices that are used for very low lifts and are not considered practicable for water works. For low lifts a screw pump also is sometimes used; and for wells, a form of bucket pump especially adapted to the confined space available.

Pumps are driven by engines operated by steam; by internal combustion of gas, gasoline, or other oil; by electricity, or by water power. The steam may cause reciprocating motion to a piston in a cylinder, or rotary motion by impact on the circumference of a revolving wheel. Internal-combustion engines produce reciprocating motion only. Electricity and water power are used to produce rotary motion only.

A reciprocating engine may be directly connected to a reciprocating pump, and a rotary engine to a rotary pump. Or the connection may be indirect, through a walking-beam, by means of gearing or belt, etc. Or a reciprocating engine may be used to drive a rotary pump, or a rotary engine to drive a reciprocating pump. Direct connection eliminates the loss occasioned by friction in indirect connections; but it requires, in the case of

reciprocating pumps, that both the motor piston and the water piston, plunger, or bucket move at the same rate and over the same space; or, in the case of rotary motor and pump, that both revolve at the same speed; which may offer serious disadvantages, or at least cause more loss of efficiency than that caused by friction in indirect connections.

RECIPROCATING PUMPS

A reciprocating pump consists essentially of a water chamber in which a closely fitting plate (piston) or cylinder (plunger) moves back and forth. In Fig. 49 these two types are shown diagrammatically; actual applications being illustrated by the

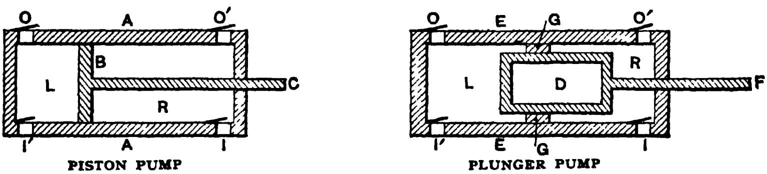


FIG. 49.—Piston and Plunger Pumps.

cylinder sections in Fig. 50 and 51. In Fig. 49, *B* represents a piston moved by a piston rod *C* in a water cylinder *A*; and *D* is a plunger (made hollow to reduce the weight) moving through a closely fitting ring *GG* in the cylinder *E*. Each cylinder is provided with openings for admitting water (*I*, *I′*) and others for its exit (*O*, *O′*), which are closed by valves. As *B* or *D* advances to the left the water in *L* is driven out through the discharge valve *O* and drawn in through the suction valve *I*, *O′* and *I′* being closed by water pressure. When *B* or *D* moves to the right, water enters through *I′* and is driven out through *O′*. *I* and *I′* connect with a suction pipe, *O* and *O′* with a discharge pipe. In most cases the level of the water to be pumped is below the cylinder, and a partial vacuum is created by the piston or plunger and the water is forced into the water cylinder by atmospheric pressure.

If the pump has two sets of inlet and outlet valves, as in the illustration, water is forced out during motion in each direc-

tion, and the pump is called a double-acting one. But if the right-hand end of the cylinder is omitted, water is discharged only during motion toward the left, and the pump is single-acting. When the cylinder axis is horizontal, as shown, the pump is called horizontal; if the cylinder stands with its axis vertical it is called a vertical pump.

In a single-acting pump, water is moving in the discharge pipe during only half the time and comes to rest during the return stroke. This is very objectionable, for it means that the inertia of the water in the pipe must be overcome anew with each stroke; also that a pulsation and " water-hammer " are caused in the discharge pipe and all pipes which it feeds. To reduce this feature, two, three, or even four such cylinders may be connected to act in rotation, so that a continuous flow of water is produced. If the pump is double acting, water is discharged during both strokes; but there is an instant at each end of the stroke when no water is being discharged, and during the stroke the rate of discharge may not be uniform. For this reason both economy of power and avoidance of pulsation of water are secured by operating two, and sometimes three, double-acting cylinders as parts of one pump, so connected that the maximum discharge of one occurs during the minimum discharge of the other. Pumps with two or three cylinders, whether single or double acting, are called duplex or triplex respectively.

But no matter how many cylinders be used, the flow is not perfectly uniform. If an air chamber be connected to the discharge pipe the air will act as a spring or cushion, compressing under the higher pressure and giving out pressure under the low, and thus equalizing the pressure in the discharge main. When a single cylinder in used, either single- or double-acting, such an air chamber is an absolute necessity to prevent destruction of pipe joints and even of the pump, and it is desirable in connection with any pump connected to a long delivery pipe. Small air chambers are often made of cast iron as part of the pump (see Fig. 50); but the larger ones are preferably in the form of a tank of steel plate. They are always set above the discharge pipe outside of the check valve, when there is one

(as there generally is), and should be filled with air at the minimum pressure experienced during operation of the pump.

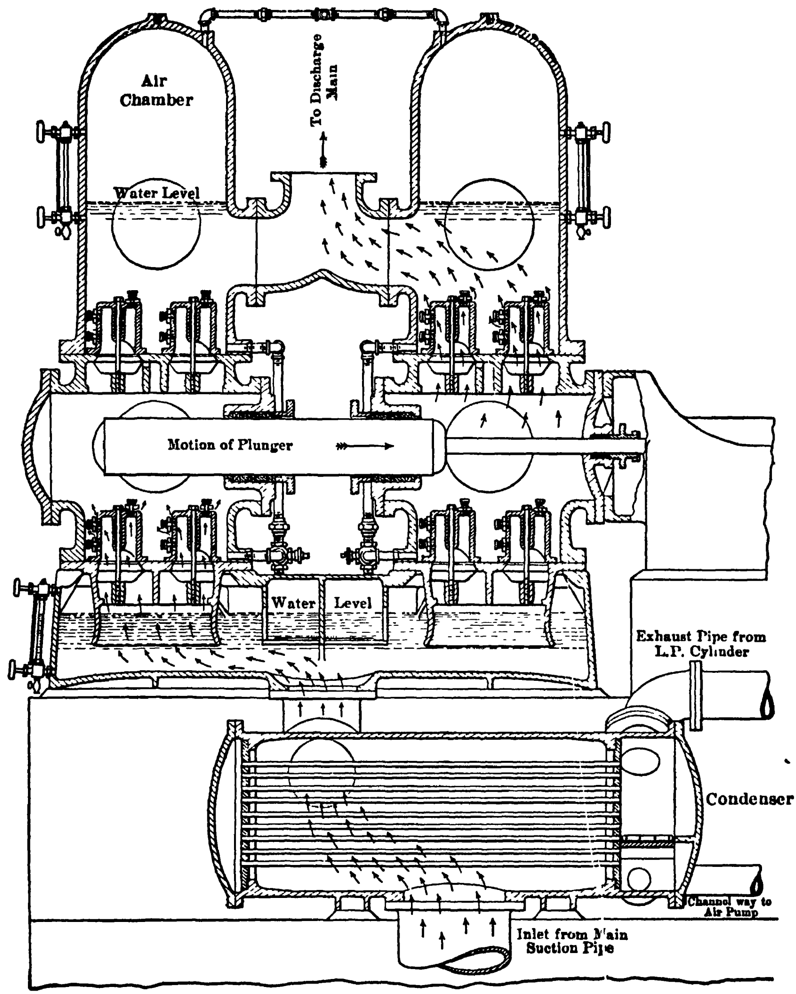


FIG. 50.—Water Cylinder with Center-Packed Plunger.
Allis-Chalmers horizontal, cross-compound, crank and fly-wheel pump.

As to size, a more or less arbitrary rule is to make the diameter $1\frac{1}{2}$ times that of the force main, the height $7\frac{1}{2}$ times, and the opening into the bottom of the chamber one-half such diameter.

The piston is adjusted to the cylinder, or the plunger to its plunger sleeve (plungers are almost universally used now), with as water-tight a fit as is possible without causing loss of energy by friction; but under the pressure produced in the water there is apt to be a slight leakage back into the other end of the cylinder; and this increases with use as the rubbing surfaces wear slightly. Moreover, at the reversal of stroke, pressure suddenly ceases in one end of the cylinder and changes to suction, while suction in the other end changes to pressure. This causes the valves that were open to close, but during the closing some water passes back through them. To hasten the closing, springs are used to seat the valve quickly;

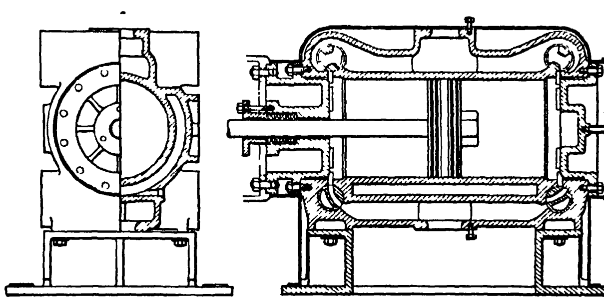


FIG. 51.—Steam Cylinder of Pump Shown in Fig. 50.

but at the best, some water escapes through them in the reverse direction. This backward movement of water through plunger ring and valves is called the "pump slip." It is seldom if ever less than 1 per cent of the amount pumped, is about 5 per cent in a reasonably good pump, and when the plunger and ring are badly worn, valves or valve springs broken, etc., it may reach 50 per cent or more. A great advantage of the single-acting plunger pump is that the plunger ring may be packed, or the fit regulated, from the outside without taking the pump apart, and thus the plunger slip kept at a minimum; it also being possible to see at all times how much of such leakage there is. (The same advantage is obtained in some recent double-acting pumps by center packing and using two water cylinders with one plunger, as in Fig. 50.)

In addition to slip, there is some loss of power due to friction in the cylinder and of the plunger rod in its stuffing box, to the opening of the valves, and operating other minor moving parts. This reduces the efficiency of the pump as a piece of mechanism. The "mechanical efficiency" of a pump is the quotient of the amount of power developed in lifting water divided by the power applied to the plunger rod or to the crank shaft to which the rod or rods are attached; both expressed in foot-pounds per unit of time. The mechanical efficiency of high-class double-acting pumps generally ranges from 85 to 97 per cent, the latter being

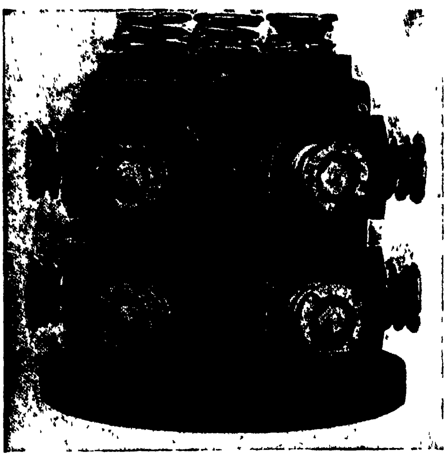


FIG. 52.—Pump Valve Cage and Valves.

exceptionally high and probably not attainable under actual working conditions.

Another type of pump consists of a vertical cylinder with one inlet valve in the bottom and another in the piston, while the outlet valves are on the top of the cylinder. This is called a bucket pump (see Fig. 53). It is especially adapted for use as a deep-well pump, since the entire cylinder can be dropped into a well, there being no openings or moving parts on the sides, and the up and down motion of a vertical rod supplying all the power required. Also the pump is extremely simple in construction. As ordinarily made, its mechanical efficiency is low.

If the piston rod, or that portion entering the cylinder, is of

considerable size, the pump will discharge water during both the down and up stroke, the former by the displacement of a volume equal to that of the piston rod, the latter by direct lift by the piston. Such a piston is called a differential plunger. (See Fig. 54.)

Vertical cylinders on large engines have been supplied with differential plungers, taking the water through one bottom port only; but the common practice for vertical pumps is the single-

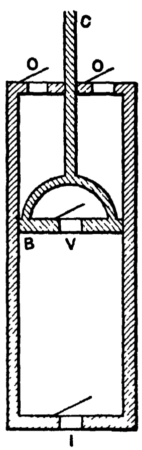


FIG. 53.—Bucket Pump.

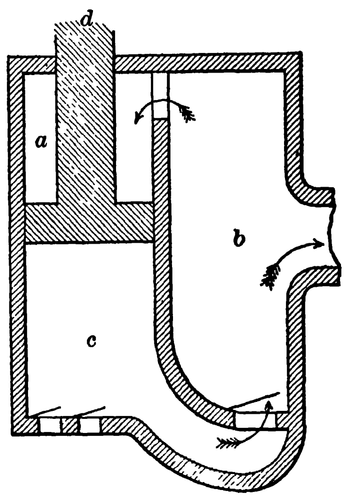


FIG. 54.—Differential Plunger. Area of plunger d is about half that of cylinder c. When the piston descends, water in c is forced into b, half passing into the main and half entering a; the latter being forced into the main on the up stroke.

acting plunger, generally triplex in pumps of 1,000,000 gallons per day capacity or larger.

The force required to pump is that necessary (1) to raise the water by suction into the cylinder, (2) to raise it by pressure against the head of water above it, (3) to overcome the friction in the pipes, (4) the inertia of all the water moved, (5) the friction and inertia of the moving parts of the pump, and (6) the friction of water in passing through the pump. The first three are useful work performed by the pump and credited to it. The

others are energy used up by the pump itself and to be charged against it. The work performed in suction and overcoming pressure head is the product of the weight of water lifted by the distance through which it is lifted (or its equivalent in pressure pumped against). The friction in the pipes leading to and from the pump can be learned only by measuring by pressure gages, although it can be calculated approximately.

All the other uses of the energy are losses that are due to the pump itself. If the pump is single acting, the water in the discharge pipe (possibly thousands of feet long) comes to a partial rest at the end of each stroke and considerable energy is required to overcome the inertia at the beginning of the following stroke. If the pump gives a continuous flow, this loss is eliminated. Friction of water in passing through the pump can be reduced by causing it to follow a path of easy curves and with as little change in cross-section as possible. To effect the latter, the entrance into the cylinder is made through an area of valve opening a little larger than that of the suction pipe or cylinder. To have one valve of this area and which would open sufficiently wide and close quickly would be impracticable, in the case of a large pump, so a large number of smaller valves is used for both intake and discharge. (See Fig. 52, showing a group of valves on a cage.) To reduce the power required to overcome inertia in the moving parts of the pump, these must be light in weight, and their motion slow (at least at the beginning and end of the stroke), and the frequency of stopping and starting should be a minimum. The latter is met by a long stroke, the longer the better. The slower the speed the larger the plunger required for a given rate of discharge. The length of stroke and size of plunger are limited by practical considerations, including cost.

As stated before, the losses in and by the pump, including slip, will ordinarily lie between 3+ and 15 per cent of the power used by it in the case of a higher-grade pump; and be much more in a cheaply or unscientifically constructed one.

A double-acting plunger working in a single chamber (Fig. 49, *d*) passes through a sleeve in the center of the cylinder, and

this may be of either the ring or inside-packed type. In the former a bronze ring is fastened to an annular flange in the cylinder, the ring fitting the plunger exactly. Wear between ring and plunger can be remedied only by replacing the ring, but with water without sediment (which is the only kind water works pumps should receive), this wear is very slow. In the inside-packed plunger, a stuffing box is substituted for the ring, the advantage of which is that any wear can be taken up by adjusting the stuffing box, but with the disadvantage that the packing wears and leaks much more rapidly than a ring, and, the stuffing box being out of sight inside the cylinder, this condition may remain unknown for a long time. For clear water the ring is believed to be preferable to the inside packing.

For a single-acting plunger, the sleeve is at the end of the cylinder, and for these the stuffing box is decidedly preferable to the ring, giving the "outside-packed" plunger. In this type, any leakage through the stuffing box is plainly visible and can be remedied at once without stopping the pump.

A double-acting plunger operating in a pair of chambers, as in Fig. 50, gives opportunity for "outside center packing" which gives the advantage of evidence of leakage and accessibility possessed by the single-acting plunger combined with the capacity and other advantages of the double-acting.

CENTRIFUGAL PUMPS

Until comparatively recent years, centrifugal pumps had been used for low lifts only and were of low efficiency; but they are now coming into increasing use for heads up to 350 feet, and in some cases for heads of 900 to 1000 feet. A low-lift pump consists of a casing having the form of a volute, in which revolves a set of curved arms (called an impeller), that fit closely the inside of the casing, the shell being open in the center on one or both of the sides, through which opening the water enters from the suction pipe; and the water leaving through an opening in the circumference. When the impeller is revolved, the water in the casing is carried around by it and the centrifugal force causes it to leave through the outlet *O*, Fig. 55, and at

the same time produces pressure in the discharge pipe if resistance is offered. This departure of the water tends to produce a vacuum in the casing and causes water to enter at the center *I*. The flow is continuous (not intermittent as from a reciprocating pump), and there is no inertia of water in discharge pipe or of moving parts to be overcome after the pump is in motion at full velocity. The slip, however, may be considerable, reaching even 100 per cent if the velocity of revolution be too low compared to the head to be pumped against, or if impeller blades or casing become too much worn. Such pumps are subject to little wear, however, unless the water contain considerable grit, the only rubbing parts being at the bearings of the impelling shaft.

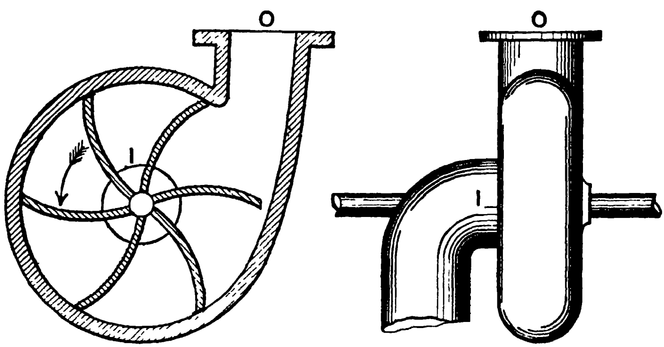


FIG. 55.—Centrifugal Pump.

The ordinary centrifugal, single-impeller, volute pump is not adapted to work efficiently against heads of more than 150 feet, although some have given satisfactory service under double this head. For heads even less than 150 feet and especially for higher ones, instead of a volute casing a round casing is more efficient, resembling that of a turbine water wheel, because of which resemblance such pumps are generally called turbine pumps.

For very high heads, a multi-stage pump is used, this consisting of a series of turbine pump casings connected together, each casing having its impeller, the several impellers being all driven by the same shaft. Each intermediate casing or stage draws its suction from the discharge of the preceding one and in

turn discharges into the next, the pressure in the water thus being raised progressively. In good practice, each stage increases the pressure about 45 to 65 pounds, although 85 pounds has been attained; but the velocity necessary to raise the pressure more than 65 pounds is likely to cause pitting of the vanes or excessive wear on the impeller and diffusion tips.

No head or pressure should be lost by shocks or abrupt changes of velocity. In multi-stage pumps, each stage is fitted with a whirlpool chamber or a device with divided stream passage known as diffusion vanes, which guide the water to the discharge opening and permit it to convert its velocity into pressure with as little friction as possible.

A pump casing must be full of water before any suction can be exerted by a revolving impeller, and must therefore be primed before it is started. In some cases the pump is set with the casing horizontal and shaft vertical and the entire casing below the level of the water, and the pump is thus always primed. (The pump need not be *in* the water, but the water can flow to it through the suction pipe; which construction permits access to the pump at any time for inspecting or adjusting it.) Where it is set above the water, however, either there must be a check valve or foot valve in the bottom of the suction pipe, and the pump is primed by filling it with water from above; or it is filled by producing a vacuum in the casing with an air pump and thus sucking water into it from below. On large pumps foot valves should not be used, as the sudden closing of the foot valve, as by a sudden stopping of the pump, may cause a shock that will split the pump casing or piping. A foot valve should have at least $1\frac{1}{2}$ times the area of the suction pipe, to minimize the friction of the water passing through it.

The discharge pipe should be the same size as the suction, and connected with the pump discharge by a tapered section. On large pumps, and all acting against any considerable head, there should be a valve on the discharge pipe, which should be closed when the pump is started, and, when the impeller has been speeded up, should be opened very slowly, allowing the full load to come upon the motor as gradually as possible. A check

valve also is generally placed on the discharge if the head exceeds 30 feet. When there is a foot valve, the pump can be primed by connecting the discharge pipe, above the check or valve, with the suction pipe by a small by-pass.

The efficiency of a centrifugal is the quotient obtained by dividing the work done in lifting water against the total suction and discharge head, by the foot-pounds applied to the impeller shaft. High efficiency involves changing the kinetic energy of the water, as it leaves the impeller, into pressure with a minimum of loss of energy by water friction (as by churning in the chambers), by friction of water on the impeller, and by friction of bearings. The most serious loss is that of friction between the impeller and the water. The wasted energy varies as the cube of the head pumped against. Also, as this head increases, the loss from friction on the impeller increases at a more rapid rate; and thus a limit is placed on the head against which the pump can be operated economically. The highest efficiency obtained with large pumps is 90 to 92 per cent, and it may be as low as 30 to 40 per cent. In multi-stage turbines, 85 to 87 per cent is about the highest efficiency reached. For the latter pumps the following are considered high efficiencies: 10,000 to 6000 gallons per minute—75 to 78 per cent; 6000 to 3000 gallons—73 to 75 per cent; 3000 to 900 gallons—70 to 73 per cent; 900 to 250 gallons—70 per cent; under 250 gallons 55 to 65 per cent.

Speeds for small pumps vary from 3500 revolutions per minute for the smallest, down to 600 for 8- or 10-inch pumps, while large pumps may run as slowly as 150 revolutions with good efficiency. (The " size " of a pump is that of its discharge pipe.)

Centrifugal pumps are specially adapted to moderate and large quantities of water rather than small. Their special features are simplicity of construction, wide passageways, absence of valves, low first cost, light weight and small space occupied relative to capacity, and adaptability to direct driving by electric motors and steam turbines. They possess the advantage that it is impossible for them to produce a pressure in the pipe lines higher than that corresponding to the speed, obviating danger

of a break in the pipe system when a valve or fire hydrant is closed quickly, which is especially important in connection with high-pressure fire service. On the other hand, should a break occur in the pipe line, reducing the pump head and increasing the quantity capacity, the pump may overload the motor unless care is taken in designing the impellers to prevent this.

If the head pumped against is greater than that for which the pump was designed, the quantity of water discharged will be less, and may be reduced to zero under very excessive heads.

Turbine pumps are used for water works pumping in a rapidly increasing number of cities, with capacities for each pump of one million to thirty million gallons per day. Also for central fire stations with capacities of 3000 gallons per minute against 300 pounds pressure (Brooklyn, N. Y.), down to small "underwriter" pumps of 500 gallons against 125 pounds. Underwriter fire pumps are made of four standard sizes: 500, 750, 1000 and 1500 gallons per minute.

SCREW OR PROPELLER PUMPS

In a screw pump the water flows axially and the pressure is not obtained by centrifugal force, but by impact or shock between propeller and water. Such pumps may be used for moving large bodies of water under low heads. About their only use in water works plants would be for raising water short distances into sedimentation basins or purification plants; but even for this the centrifugal pump is preferred.

SUCTION LIFT

In any type of pump, if the pump chamber is higher than the water to be lifted, the water is raised into it by the pressure of the outside air when a partial vacuum is created in the pump. If a perfect vacuum were created the water would rise through a height of 34 feet. But this is not quite possible. Moreover a part of the 14.7 pounds air pressure must be used in overcoming friction of the water in entering the suction pipe (this can be reduced by making the end of the suction pipe bell-mouthed), in passing through it, and (in the reciprocating pump) in lifting the valves and passing through the valve openings, and in filling the cylinder behind the plunger. There

should also be a factor of safety. It might be possible under favorable conditions to lift water 26 feet, 8-foot head being used in overcoming these resistances; but there is an undesirable risk in making the suction lift more than 20 feet; and it is desirable to keep it down to 12 or 15 feet if the suction pipe is short, and to still less if it is long. If the water is to be carried more than 50 feet to reach the pump, a suction well should be built under or near the pump and the water be led into this and the pump suction draw from it. For a 20-foot lift it is desirable that the suction pipe be 10 inches for a one-million gallon pump (capacity 1,000,000 gallons per twenty-four hours); 20 inches for a five-million, and 30 inches for a ten-million gallon.

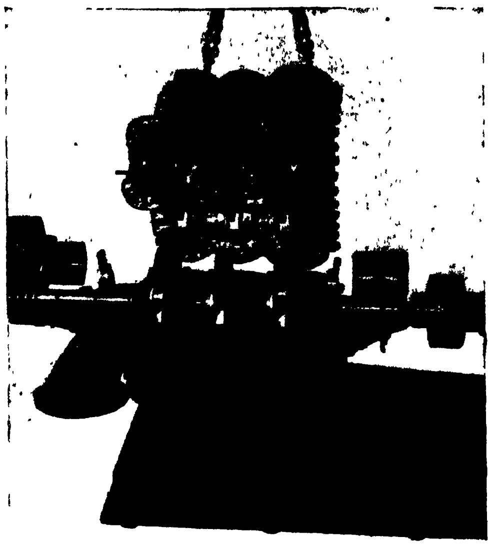
Art. 59. Motive Power for Pumps

Steam power is used for more water supply pumps than all other forms of power combined; but quite a number are operated by internal combustion engines, others by electric motors, and a few by hydraulic turbines.

Centrifugal pumps are generally driven by a steam turbine, or electric or water motor, mounted upon the same shaft as the pump, or by belt or gears from one of these or from a steam or internal-combustion engine. In the latter case any speed of pump can be combined with any speed of engine by proportioning the gears or belt pulleys; but when connected on the same shaft, both must have the same speed, and each should be designed for maximum efficiency at that speed.

In the case of electric motors, if the head on the pump is constant, giving constant speed, this can be secured without difficulty; but if the head varies, a motor should be used that permits speed regulation. If an induction motor is used, the pump should have a horse-power nearly constant within the limits of the working conditions, as both power factor and efficiency of such motors are reduced at light loads. There are some objections to using a synchronous motor, which is adapted to slow-speed work (below 600 r.p.m.), but in which the speed is fixed, the principal difficulty being experienced in starting the pump. Such motor is selected for the maximum

load. But any type of motor can be used successfully where the pump delivers at a uniform rate against a constant head (as



Same as below, with Top of Pump Casing Raised, showing Interior Construction.

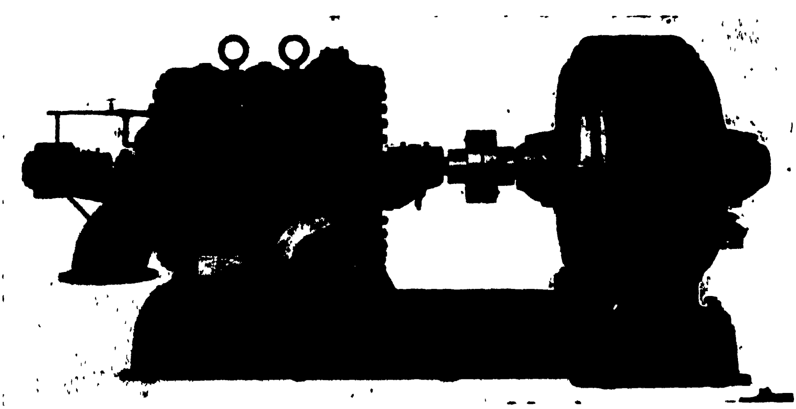


FIG. 56.—Twelve-inch Three-stage Centrifugal Pump, Direct Connected to Turbine.

Installed at Charlotte, N. C., by Morris Machine Works.

into a reservoir). Knowing the speed at which the pump is intended to revolve under ordinary working conditions, and the

horse-power required to operate it, a motor is selected designed for the same speed and delivering the required horse-power. Where it is desired to pump against two different heads, as where a "fire pressure" is used higher than the domestic pressure, a two-speed motor will give higher efficiency, but will cost more, occupies more space, and has more parts to get out of order. In one plant, for instance, a two-speed 50- and 75-horse-power motor cost $875, while a 60-horse-power constant-speed motor cost $583; each of these being direct-connected to a deep-well multi-stage centrifugal pump in a 20-inch well.

A centrifugal pump may be driven directly by a steam engine but a low-velocity pump must be used, as more than 600 r.p.m., or at most 800, is not generally practicable for a steam engine; and even this rate cannot be reached by the engine until the pump has been operating for several minutes. By using a belt or gear connection, the most effective velocity of both engine and pump can be adopted, and generally this will give a higher efficiency, even after deducting for loss by friction in gears or in belting and shafting.

INTERNAL-COMBUSTION ENGINES

Small steam-pumping plants are extravagant of fuel, and do well to secure a pump-horse-power hour on 4 or 5 pounds of coal. A good gasoline engine will generate a horse-power hour on $\frac{1}{8}$ gallon of gasoline. With coal at $3 a ton, the former gives 0.6 to 0.75 cent per horse-power hour; the latter gives 2 cents with gasoline at 16 cents per gallon. But the steam plant requires an engineer and fireman in constant attendance, the additional first cost of the boiler and boiler room and coal shed, and the additional space for these; while the gasoline plant requires practically no attention; none, in fact, except that there is a certain liability to "engine trouble," which is the chief objection to internal-combustion engines.

Internal-combustion engines are used as prime movers up to 75 h.p. with gasoline or illuminating gas; to 200 h.p. with producer or natural gas; and up to 500 h.p. with oil engines. Gasoline is changed to gas before being used in the internal-

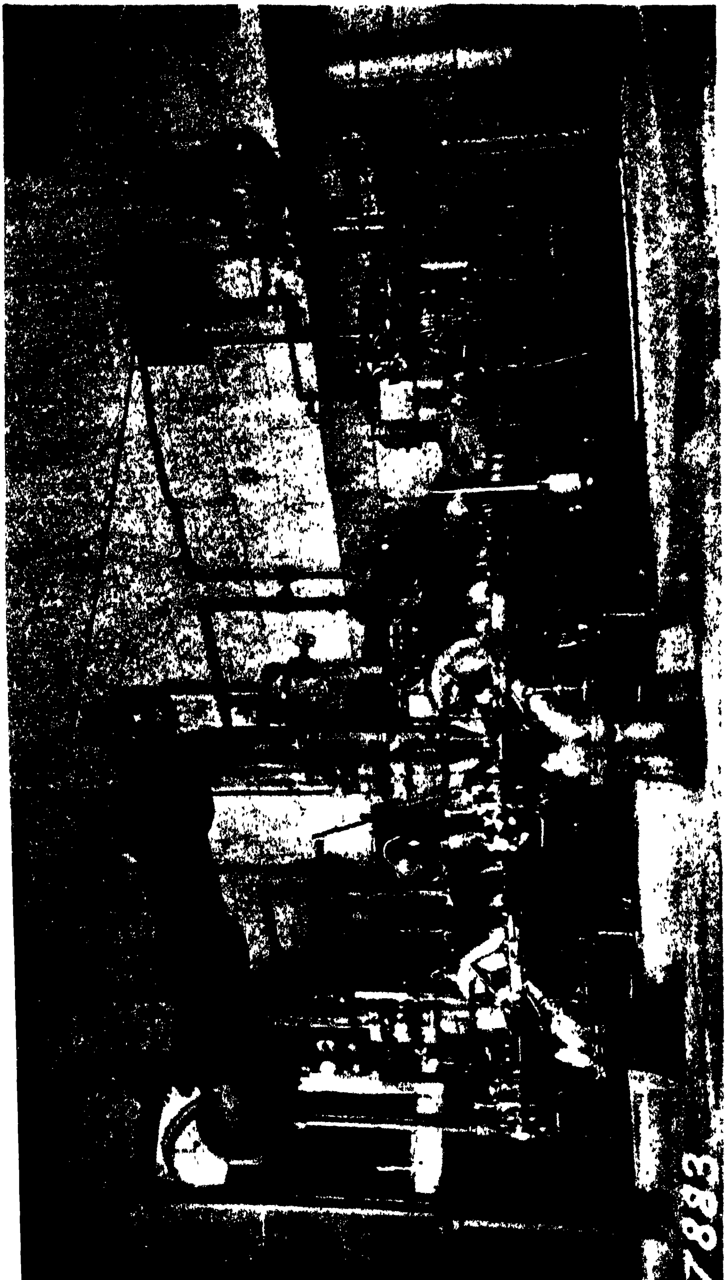


FIG. 57.—Eight-inch Three-stage Centrifugal Pumps, Direct Connected to 135 Horse power Gas Engines

Installed in Stratford, Ont., Water Works; Morris Pump; Van Blerk Gas Engine

combustion engine; and this or other gas is drawn into the cylinder of such an engine, mixed with air and ignited by an electric spark, thus producing an explosion and sudden rise of pressure. In the oil engine the fuel is introduced gradually and burns without explosion, the heat expanding the air. The air in the cylinder being compressed to 500 pounds pressure, the tempera-

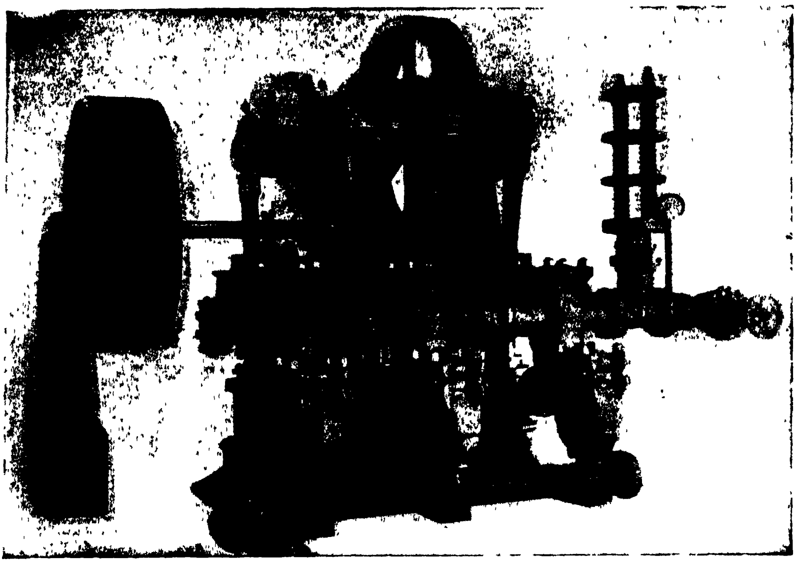


FIG. 58.—Triplex Power Pump.

Belt-connected to engine. The belt-driven wheel at the left, through shaft and pinion, drives the large gear wheel, which in turn gives reciprocating motion to the plunger rods. Made by Goulds Mfg. Co.

ture due to this compression ignites the oil. Gas engines lose power with altitude above sea level, owing to the less amount of oxygen per foot of air. For instance, an engine at Denver would have but 80 per cent as much horse-power as at sea level.

Internal-combustion engines are generally connected to the pump end by gears, sometimes by belting. A common combination is a gas or gasoline engine gear-connected to a triplex, single-acting power pump, the three plungers of which are connected to a single shaft with three cranks making angles of 120° with each other.

HYDRAULIC TURBINES

A water turbine or other water-power engine may be used direct-, belt- or gear-connected to centrifugal or reciprocating pumps, where water power is available. In case of direct connection, both pumps and turbine must of course be designed for the same rate of revolution. Efficiency of a turbine of 80 per cent may readily be obtained; but if gearing be interpolated between this and the pump, the power delivered may be only 65 per cent of that in the penstock. Water power is often subject to interruption by droughts or floods, and an auxiliary plant operated by steam, electricity, etc., should be provided for occasional use. A common combination is a turbine direct-connected to a centrifugal pump, the shaft common to the two being either horizontal or vertical.

STEAM TURBINES

Steam turbines have come into use since 1910, and are now in use in at least twenty-five water-works plants. They consist of a number of wheels or rotors on a shaft inside a casing, each carrying buckets into which steam is discharged from nozzles, the steam expansion taking place only partially in the first stage and being continued in successive stages until it finally exhausts into a vacuum of about 28 to 29 inches. They are ordinarily used for driving centrifugal pumps, but the wheels or runners make several thousand revolutions a minute. and it is therefore impracticable to connect them directly to the pumps. But reduction gears for connecting the pump and steam turbine through flexible couplings have been perfected that give 96 to 98 per cent efficiency.

Any steam pressure can be employed, and in some cases steam turbines are driven partly or wholly by the exhaust from a reciprocating engine. They also utilize efficiently any degree of superheat. For general water-works practice the best results are obtained with 175 pounds steam pressure, 100° superheat, and 28½ to 29 inches vacuum.

Given a vacuum of 28 inches and 150 pounds steam pressure, with dry steam, the pounds of steam required per pump horse-power per hour will vary from about 16.5 for 100 h.p. to 13.2 for

TABLE No. 36

PERCENTAGES OF WATER WORKS PLANTS USING EACH OF THE SEVERAL KINDS OF POWER FOR PUMPING. BY GEOGRAPHICAL DISTRICTS

(Compiled from information from 609 cities by "*Municipal Journal*" in 1917

Geographical Division.	Percentage Plants in this District are of all the Plants Considered.	Steam.	Electricity.	Internal Combustion.	Hydraulic.	Steam and Electric.	Other Combinations.
New England	9.85	53.3	13.3	10.0	5.0	8.3	10.0
Middle Atlantic	13.80	57.1	9.5	4.8	1.2	11.9	15.5
East North Central .	31.00	55.0	15.9	3.2	2.1	19.0	4.8
West North Central	17.60	41.1	33.6	1.9	. .	21.5	1.9
South Atlantic. .	9.03	52.7	27.3	3.6	.	12.7	3.6
East South Central	6.24	68.4	21.1	2.6	.	7.9	
West South Central. .	5.75	51.4	14.3	8.6	.	22.9	2.8
Mountain. . . .	1.64	10.0	70.0	. .	.	20.0	
Pacific.	5.09	16.1	51.6	3.2	. .	9.9	19.2
Entire United States. .	100.00	50.4	21.9	4.1	1.3	15.9	6.4

Same as above, but with figures for each kind increased by adding to them as follows: Where two kinds of power are combined in one plant, each kind is credited as ½, where there are three kinds, each is credited with ⅓.

	Steam.	Electricity.	Internal Combustion.	Hydraulic.
New England . .	58.9	21.4	13.0	6.7
Middle Atlantic	65.7	20.4	9.7	4.2
East North Central	66.3	26.1	3.7	3.9
West North Central .	52.3	44.9	2.8	
South Atlantic.	59.1	35.5	5.4	
East South Central	72.4	25.0	2.6	
West South Central. . .	64.3	25.7	10.0	
Mountain.	20.0	80.0		
Pacific.	21.0	66.1	6.5	6.4
Entire United States. .	59.5	31.9	5.8	2.8

800 h.p. The amount would increase 5 per cent for 1 inch drop in vacuum, $1\frac{1}{2}$ per cent for each 10 pounds drop in steam pressure, and 1 per cent for each 10° superheat.

Turbine pumping units (steam turbine and pump combined) have developed duties up to 152,000,000 foot-pounds per 1000 pounds of steam, and 140,000,000 is not uncommon. Such units require small space and light foundations, a less efficient and expensive engine man can be used than for reciprocating engines, and cost of repairs is low, 2 per cent of purchase cost per year being considered sufficient to cover this. Those in service vary in capacity from 100,000,000 gallons per day against a 56-foot head, in Pittsburgh, down to 2,000,000 gallons and perhaps less. The cost of the turbines in 1914 varied from $19 per horse-power for 250 h.p. to $9 for 1000 h.p. or larger.

A 1000-h.p. turbine and 12 m.g.d. (million gallons per day) pumping unit occupies 19 feet by 6 feet 9 inches, and 7 feet height.

WINDMILLS

For very small plants windmills are sometimes used, in connection with a tank for storage during calm weather. It is very desirable, however, to provide a gasoline or similar engine to supplement the wind engine. The whole may be combined in one structure, the windmill above the tank, the gasoline-engine housed in beneath it. Wind engines may be obtained of 4 h.p. easily capable of pumping 200,000 gallons per day 100 feet high; but dependence probably should not be placed upon an average of more than 75,000 gallons per day from such an engine, owing to the uncertainty of the wind.

BOILER PLANTS

Boilers in common use in water works may be classified as return tubular, set in brick work; internally fired (marine type) fire tube; and water tube. In the first the water is contained in the steel casing or shell, and the fire on the grate is immediately under one end of this shell, the flame and hot gases following along the under side of the boiler to the rear, then up, and back to the front through a large number of hori-

zontal steel tubes that connect the front and rear ends of the boiler and are surrounded by the water, the hot air passing

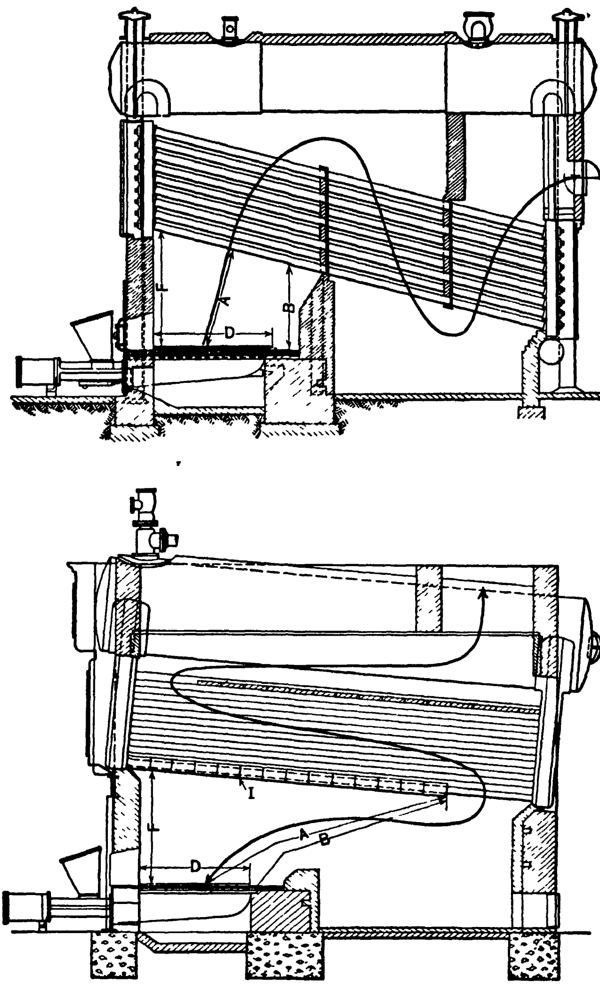


FIG. 59.—Water-tube Boilers.

Upper, Babcock & Wilcox; lower, Heine. Underfeed stoker shown in each. *D*—depth of grate. *A*—distance from center of fire to nearest exposed part of tubes.

through them thus helping to heat this water. These boilers are set in brick work that must be built on the spot. In the internally fired boiler the fire box is built of steel as a part of the

boiler and the brick setting is not necessary, but the total cost is about as great. These are not often used for permanent plants. In the water-tube boiler there is no steel shell in contact with the fire, but the water passes through several connected tubes, a large number of which are assembled in an inclined group that deliver the steam into a drum to which they are connected at their upper ends; the flame and hot gases passing up and around and between these tubes (see Fig. 59). The danger of explosion is less in these than in flue or fire-tube boilers, since tubes are not injured by intense heat as easily as riveted plates. These boilers are to be preferred for pressures of 150 pounds and over, and are common of sizes up to 300 to 500 h.p. Steam can be raised in them more quickly than in return tubular, but it requires closer attention to keep the steam pressure uniform.

The capacity of a boiler may be expressed in square feet of heating surface, but "boiler horse-power" is the unit more commonly used, this being standardized by the American Society of Mechanical Engineers to mean the power to evaporate 30 pounds of water per hour at a gauge pressure of 70 pounds and from a temperature of 100° F. This is about equivalent to evaporating $34\frac{1}{2}$ pounds of water from a feed-water temperature of 212° to steam at the same temperature. This requires about 10 or 12 square feet of heating surface per horse-power. It is better to purchase boilers by the area of heating surface, calculating the area desired on that basis.

The amount of water evaporated per pound of coal under best working conditions is about 9 pounds by the best bituminous, 8 pounds by ordinary bituminous or large anthracite, and 7 pounds by buckwheat anthracite. For ordinary bituminous the grate surface should be about $\frac{1}{48}$ of the heating surface; for large anthracite $\frac{1}{32}$, and for buckwheat $\frac{1}{28}$. With forced draft the bed of coals can be kept thicker and a smaller grate area used. As an average, where conditions are not exactly known, $7\frac{1}{2}$ pounds of water per pound of coal, and a grate area $\frac{1}{40}$ the heating surface are often used. The accompanying table shows ordinary dimensions of return tubular boilers (also called flue boilers)

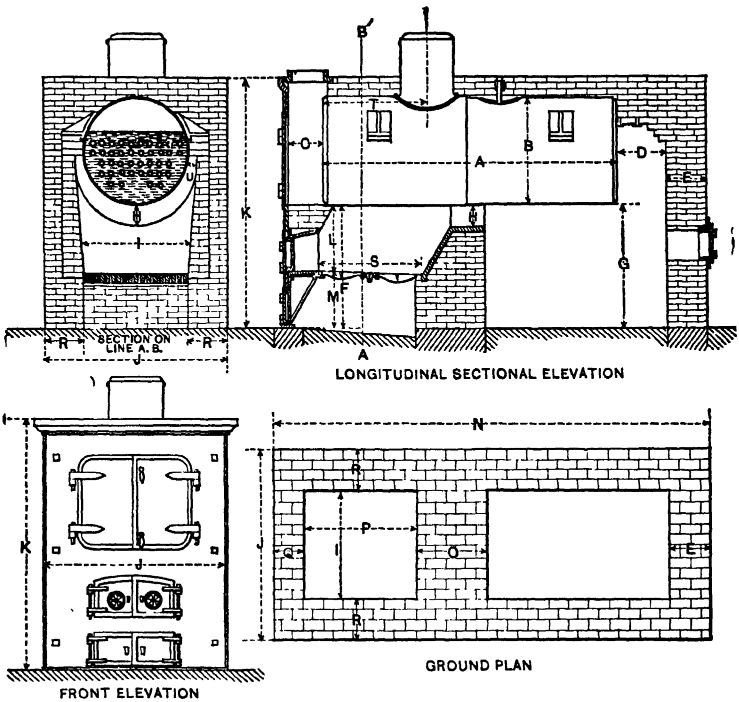


Table No. 37

MEASUREMENTS FOR SETTING OF RETURN TUBULAR BOILERS

No.	Reference Letters on Diagrams.												
	A Feet.	*B* Ins.	*C* Ins.	*D* Ins.	*E* Ins.	*F* Ins.	*G* Ins.	*H* Ins.	*I* Ins.	*J* Ins.	*K* Ins.	*L* Ins.	*M* Ins.
1	7	30	12	20	16	45	44	7	30	62	83	26	19
2	8	36	12	20	16	48	47	8	36	68	92	26	22
3	10	42	14	20	16	48	47	8	42	74	98	27	21
4	12	44	14	24	16	48	46½	10	44	76	100	27	21
5	14	48	16	24	16	47	45½	10	48	88	103	26	21
6	12	60	18	24	20	50	48½	12	60	108	118	26	24
7	16	60	18	26	20	50	48	12	60	108	118	26	24
8	16	66	18	28	20	50	48	12	66	114	124	26	24
9	18	72	20	30	20	50	48½	12	72	120	130	26	24
10	18	84	20	30	20	50	48	12	84	132	142	26	24

No.	*N* Ft.	Ins.	*O* Ins.	*P* Ins.	*Q* Ins.	*R* Ins.	*S* Ins.	*T* Ins.	*U* Ins.	Number of Firebrick.	Number of Common Brick above Floor Level.
1	11	6	20	40	12	16	36	34	4	500	4,800
2	12	6	24	40	12	16	36	34	4	600	6,000
3	14	8	28	46	12	16	42	42	4	750	8,000
4	17	0	32	52	12	16	48	49	4	900	10,400
5	19	2	36	58	12	20	54	84	4	1000	13,800
6	17	10	32	50	16	24	48	49	4	1300	17,000
7	22	0	40	56	16	24	54	96	4	1400	21,000
8	22	2	40	56	16	24	54	96	4	1550	23,000
9	24	6	40	62	16	24	60	108	4	1800	26,500
10	24	6	40	62	16	24	60	108	4	2000	31,000

In a flue boiler, the boiler shell must be sufficiently strong to resist the maximum steam pressure. In a water-tube boiler only the tubes and steam drum must resist this pressure. In the former, soot forms on the inside of the flues and scale may form on the outside; while with the water-tube the scale may form on the inside and soot on the outside.

A pumping engine is driven not by the amount of steam used, but by the amount of heat given up by the steam; consequently the hotter the steam the better. Steam is therefore "superheated" in some plants—that is, heat is added beyond that required to evaporate it under the boiler pressure. The steam may be heated 300° or more beyond the evaporation temperature, but 100° is believed to be best for pumping stations. This increases the horse-power of a boiler by 8 to 10 per cent, insures practically "dry" steam (with no entrained water) in the engine cylinders and thus reduces loss due to cylinder condensation. Superheaters are especially effective when applied in a small water-works pumping station with low-duty triple-expansion engines. The superheater consists of small tubes in coils or with return beds, heated by hot gases in the boiler furnace or from some other source. It costs about $8 or $10 per horse-power and may save 10 to 20 per cent of the fuel.

Part of the heat used in the boiler is employed in raising the temperature up to 212°, or to the higher evaporation point determined by the steam pressure; and the warmer the water that enters the boiler (called feed water) the less the heat required for this. In general, about 1 per cent of the fuel is used for every 11° F. that the feed water is heated below 212°. For this reason all waste heat about the station should be used for heating the feed water. Exhaust steam and condensation from steam jackets and from steam piping are used for this purpose, passing through a "feed-water heater." Also surplus heat in flue gases on their way from boiler to chimney may be utilized in the same way by an "economizer"—a long, narrow brick chamber through which the hot gases pass and in which is a series of cast-iron tubes through which the feed water circulates. By heating feed water in a heater and then in the economizer,

the feed water may enter the boiler at 200° or more instead of 60° and 12 to 15 per cent of the fuel be saved. Heater, economizer, and necessary pipes will cost $8 to $10 per horse-power.*

The water must enter the boiler against the steam pressure, which is generally more than 100 pounds and less than 400. If the pressure of water in the mains exceeds the steam pressure the feed water may be drawn directly from the mains. If the water pressure is less, or other water is used, a small feed-water pump is used for this purpose.

No boiler can put into the steam all the heat contained in the coal, 72 per cent efficiency being a very good record, although 81 per cent has been secured, and 86 per cent if the heat given by economizers be included. The loss of 28 per cent of heat units in the coal is due to lack of skill in firing; inability of the boiler to transmit to the water all the heat in the burning gases during the short time that they are passing through; and the heat necessary to produce draughts in the chimney. No boiler should show less than 65 per cent efficiency in practice, and more than 80° is unusually good, as from 10 to 15 per cent is required for draught.

Chimneys. The force of draft created by a chimney is proportional to the square root of its height. Strong drafts are necessary if low-grade fuels are to be used, or if a thick bed of coal is to be burned—that is, if the grate area is small. High chimneys permit "forcing" a boiler when necessary and may therefore serve instead of reserve boiler heating surface for emergencies. They also carry the smoke above housetops and thus minimize nuisance. The following are recommended for ordinary conditions: Free-burning bituminous, 75 feet; slow-burning bituminous slack, 100 feet; anthracite pea, 125 feet; anthracite buckwheat, 150. For plants of 1000 horse-power or more the range should preferably be from 150 to 200 feet.

The height being determined upon, the area may be determined by one of several more or less empirical formulas. Kent gives area $A=\frac{3P}{10\sqrt{H}}=\frac{C}{16.6\sqrt{H}}$; in which P is the horse-

* All costs in this chapter are on the basis of pre-war prices.

power of the boiler, H is the height of the chimney in feet, and C is the pounds of coal burned per hour (assumed to be 5 pounds per horse-power). The area to be used is that of a section 4 inches larger each way than that calculated, it being assumed that there is 2 inches of "dead" gas around the perimeter. Gale's formula is $A = 0.07C^{\frac{3}{4}}$. Christie gives $A = \dfrac{P}{3.24\sqrt{H}} = \dfrac{C}{13\sqrt{H}}$, 4 pounds of coal per horse-power-hour being assumed.

Chimneys are generally built of brick, concrete, or steel. They must be able to resist the strongest wind without overturning; and the foundation must be absolutely unyielding to maintain the chimney plumb, remembering that wind may

Table No. 38

BOILER HORSE-POWERS FOR WHICH VARIOUS CHIMNEY HEIGHTS AND DIAMETERS ARE BEST ADAPTED. PREPARED FROM KENT'S FORMULAS, BY CHICAGO BRIDGE AND IRON WORKS

Diam. in Ins.	Height of Chimney, in Feet.										
	50	60	70	80	90	100	110	125	150	175	200
18	23	25	27	29							
21	35	38	41	44							
24	49	54	58	62	66						
27	65	72	78	83	88						
30	84	92	100	107	113	119					
33	.	115	125	133	141	149	156				
36	..	141	152	163	173	182	191	204			
39	...	.	183	196	208	219	229	245	268		
42			216	231	245	258	271	294	318	340	364
48	..	...	.	311	330	348	365	389	428	459	491
54	...	..	..	363	427	449	472	503	551	594	635
60	...	...	...	505	539	565	593	632	692	748	797
66	..	...	...	...	658	694	728	776	849	918	981
72	...	..	...	...	792	835	876	934	1023	1105	1181
78	..	...	...	...	..	995	1038	1107	1212	1310	1400
84	...	..	...	...	...	1163	1214	1294	1418	1531	1637
90	...	...	...	...	...	1344	1415	1496	1639	1770	1893
96	..	...	...	...	...	1537	1616	1720	1876	2027	2167
108	.	...	.	..	..	...	...	...	2290	2470	2637
120	..	..	.	...	.	...	....	...	2827	3049	3255

cause a unit pressure on the leeward side double the average pressure. Brick must resist overturning by wind pressure by its weight only; steel chimneys rely largely upon tensile strength and anchorage to a heavy foundation, or in some cases upon guys. Reinforced concrete chimneys may be anchored to a foundation, and rely partly upon weight and partly upon tensile strength of reinforcement. Steel chimneys are apt to leak at the joints after some months of use, and to rust badly, and are probably more expensive in the long run than concrete or brick; but may be almost necessary where a solid foundation cannot be found to carry the greater weight of the masonry chimney; and are generally used where the plant is temporary only.

Art. 60. Pumping Engines

The majority of water-works pumps are operated by steam, and most of those so operated are what are known as pumping engines—that is, both pump and steam engine are combined in one piece of apparatus. There are several types of these, and numerous variations of each type. They may be classified according to the pump end, according to the steam end, and according to the combination of the two.

Considering the pump end, the terms single, duplex and triplex are applied according as there are one, two or three water plungers or pistons. The reciprocating part may be either a piston or a plunger; it may be single-acting or double-acting—that is, may discharge water during only one-half or during both halves of the stroke. The plunger may be either ring, inside packed or outside packed, the last applying to a single-acting plunger, or a double-acting plunger with two plunger chambers. The last two terms (see Art. 58) refer to the ability to adjust the stuffing box or packing around the plunger from the outside, or the necessity of removing the cylinder head and adjusting it from the inside.

The variations of the steam end are even more numerous. Before describing them, an explanation of the action of the steam end is necessary. The steam piston is given its reciprocat-

ing motion by admitting steam first into one end of the cylinder and then, when the piston has traveled nearly to the other end, by admitting steam behind it at this end, permitting the steam previously admitted to leave the cylinder as the piston makes its return stroke. The steam is admitted from the boiler at approximately the pressure in the boiler. If it continues to enter the cylinder during the entire movement of the piston in one direction, the pump is described as high pressure, the pressure in the cylinder at the end of the stroke being approximately the same as at the beginning, namely, that in the boiler. If, however, the steam be cut off when the piston has traveled a part of its stroke only, the steam already in the cylinder will expand and continue to force the piston ahead, although with a pressure diminishing and always inversely as the volume occupied by the steam. In this way the pressure may be lowered to a small fraction of that in the boiler.

The piston operates the pump by forcing out and back again the piston rod attached to it, and the pressure exerted upon this rod, and through it to the plunger, is equal to the area of the piston in square inches times the pressure per square inch of the steam in the cylinder. Assuming that the mechanism is devised so as to average up the pressure between the maximum delivered at the beginning of the stroke and the minimum delivered at the end, such average will, of course, be less than the maximum; and therefore to deliver a certain average pressure with a given boiler pressure, the area of piston and cylinder must be greater than in the case of a high-pressure engine. Another disadvantage of using steam expansively is that, as the pressure decreases, the temperature does also, and we have a continuous rising and falling of temperature of steam, and consequently a tendency for the temperature of the cylinder and all its parts to vary similarly. This latter effect may be counteracted to a large extent by steam-jacketing the cylinder—that is, providing a jacket or envelope around the cylinder through which steam from the boiler is passed.

The advantage of using the steam expansively is that much more of the power existing in the steam by reason of its pressure

and temperature is utilized. Still more can be utilized by condensing the steam as it leaves the cylinder (in which case it is called a condensing engine), by which condensation a partial vacuum is created. Still another method is to use the exhaust from one cylinder in another larger cylinder. For instance, if the steam in the high-pressure cylinder is allowed to expand to double its original volume, it can then be discharged into a low-pressure cylinder having double the size of the high-pressure and here be allowed to further expand until discharged at approximately air pressure, or at less than air pressure into a condenser. In high-duty engines the process is carried still further and we have high-pressure, intermediate-pressure and low-pressure cylinders receiving the same steam in succession. One considerable advantage of this double or triple expansion is that the variation of temperature in any one cylinder is only about one-half or one-third as great as when but one cylinder is used. This not only prevents rapid changes in dimensions of the metal of the cylinder, but diminishes the loss of heat by radiation from the cylinder, or, if the cylinder is steam-jacketed, diminishes the amount of heat lost in the jacketing steam.

It is found impracticable to reduce the steam pressure at the end of its work to less than 5 pounds absolute pressure, or about 10 pounds less than atmospheric pressure, and the majority of engines do not equal this. (The steam gage registers pressure above atmospheric pressure, and absolute pressure is obtained by adding to this 14.7 pounds—15 is used for approximate calculations.) If, therefore, the steam gage reads 80 pounds, indicating 95 pounds absolute pressure, and the pressure as discharged from the final cylinder is 5 pounds, the steam has expanded 19 times. If the boiler pressure had been 150 pounds, or 165 pounds absolute, the steam would have expanded 35 times, or nearly twice as much. It would, of course, have required considerable more coal to have raised the temperature of the steam from 80 degrees to 150 degrees; but with full allowance for this, we still would obtain a greater amount of energy per unit of heat (or its equivalent of coal) from a given

applicance, thus increasing the efficiency or duty of the engine. Actual figures indicate that by increasing the steam pressure in a given engine from 80 to 150 pounds and at the same time decreasing the absolute pressure at the terminal from 10 pounds to 5 pounds, the duty per thousand pounds of steam has been increased 27 per cent.

The work done by a given engine is expressed in terms of pressure exerted in the cylinder times feet over which the cylinder rod passes in a unit of time. Consequently, the greater the speed of a piston of a given size, the greater the amount of energy developed. In a reciprocating engine, all of the moving parts must be brought to a standstill and given motion in a new direction twice during each complete cycle, these including the piston and its rod, the valves controlling the entrance and exit of the steam, the water plunger, and the valves through which the water enters and leaves the cylinder. The more rapidly the piston moves, the greater the momentum to be overcome in reversing direction of motion. For this and for other practical reasons, high velocity introduces losses which, if carried beyond a certain point, are not compensated for by any saving in construction cost secured by reducing the size of the engine and pump. Low cost of a small high-speed engine should not tempt the purchaser to overlook the disadvantages of this loss of energy and the additional one that there is much greater probability of the breaking of valves and other minor parts and a general racking of the pump which cause high maintenance charges.

The varying steam pressure in a cylinder can be recorded by an indicator, a small contrivance attached to the cylinder at one end, in which contrivance a pen records upon a small revolving cylinder a curve whose vertical ordinates are proportional to the pressure in the cylinder. Cards placed on the three cylinders of a triple-expansion engine are shown herewith. In this case the initial steam pressure was 165 pounds absolute, leaving the high-pressure cylinder and entering the intermediate at 42.36 pounds absolute, leaving the intermediate cylinder and entering the low pressure at 11.45 pounds absolute, and leaving the low-pressure cylinder at

5 pounds absolute. The high-pressure cylinder had a diameter of 30 inches, the intermediate cylinder 56 inches, and the low-pressure cylinder 84 inches. All three cylinders had a stroke of 66 inches—that is, each of the pistons traveled 66 inches in each direction. The steam entering the high-pressure cylinder was cut off when the piston had traveled 16.96 inches, and during the remaining 49.04 inches exerted pressure through expan-

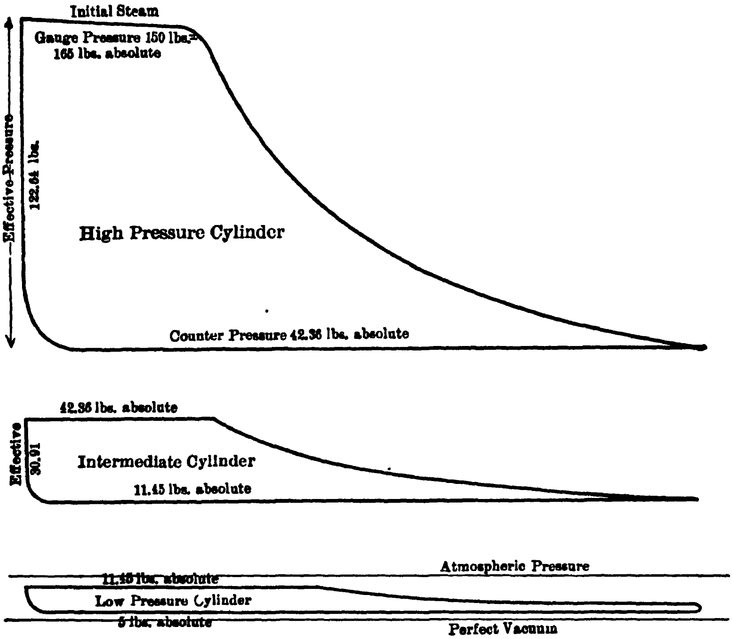


FIG. 60.—Mariotte Curve of a Triple Expansion Engine.

Range of temperature: High-pressure cylinder, initial temperature, 365°, terminal temperature, 270°; intermediate, 270° and 197°; low-pressure, 197° and 162°. Adapted from Chas. A. Hague's "Pumping Engines for Water Works."

sion only. The steam was cut off from the intermediate cylinder at 17.84 inches from the beginning of the stroke and from the low-pressure cylinder at 28.73 inches from the beginning of the stroke. As is seen from the diagram, the net or effective pressure against the piston in the high-pressure cylinder remained at $(165-42.36)=122.64$ pounds during the first 16.96 inches of the stroke, after which it decreased to zero at the end of the stroke, but more rapidly at the beginning than at the end.

If the area enclosed in this diagram be divided by the horizontal axis, or length of stroke, the mean effective pressure would be found to be 52.323 pounds. As the area of the piston was 706 square inches, the mean effective pressure exerted on the piston was 36,940 pounds. As this piston was traveling 200 feet per minute, it developed 224 h.p. A similar calculation for the intermediate cylinder shows that the mean effective pressure was 14.998 pounds, and that the area of the piston was such as to give the same mean effective pressure and consequently the same horse-power (since all three pistons traveled at the same rate) as the high pressure. Similarly, the mean effective pressure in the low-pressure cylinder was $6\frac{2}{3}$ pounds, but the mean effective force on the entire area of the piston totaled the same as for the other two cylinders. The total horse-power by the three cylinders was therefore three times 224, or 672 h.p. Calculating from the size of the cylinder, it can be determined that the amount of steam used per hour was 6334 pounds. This was a steam-jacketed engine, and 1158 pounds per hour were used in the steam jackets, giving a total amount of steam consumed per hour of 7492 pounds, or 11.14 pounds per indicated horse-power per hour.

The 672 h.p. referred to was that developed by the pistons and is known as indicated horse-power. The overcoming of inertia of the moving parts of the engine, friction of the piston and piston rod and other moving parts, and other mechanical losses, may amount to from $3\frac{1}{2}$ to 20 per cent or more, 4 or 5 per cent loss being a good record for a high-duty engine. This means that 95 or 96 per cent of the 672 i.h.p. would be actually transmitted to the pumping end of the pumping engine. The power so transmitted is called the pump horse-power.

Until the past few years, practically all pumping engines were built with the axes of both steam cylinders and water cylinders horizontal. There has been developed a high-duty engine in which the cylinders are placed with their axes vertical, called a vertical engine. In the horizontal engines, the steam and water cylinders were in most cases "direct connected," that is, the same piston rod extended from the steam piston to

the plunger or piston of the water end. In the crank and flywheel pump, of which the Holly is the best known illustration, the steam piston rod, by means of a crank or eccentric, drove a shaft carrying a large flywheel, to which was also connected the piston or plunger rod of the pump. The object of this construction was as follows: During the first part of the stroke the pressure on the steam piston is considerably greater than the mean for the entire stroke, while at the latter end it is considerably less. In order to maintain uniform rate of motion of the water plunger and also to use effectively the surplus energy of the first part of the stroke, it is necessary in some way to store up this surplus to be utilized at the end of the stroke. This is done by means of the flywheel, which, because of its mass, continues to revolve at a uniform rate and thus carries the water plunger at such rate through the latter part of the stroke when the steam pressure diminishes to zero. Another method of accomplishing the same purpose was adopted for the Worthington high-duty pump, this consisting of two plungers which compress water in two small cylinders during the first part of the stroke, which water expanding added its force to the piston rod during the latter part of the stroke.

In most of the horizontal direct-acting engines the two or three cylinders of the compound or triple-expansion engine were placed in line so that all actuated the same piston rod. In one type of pump, however, a high-pressure and a low-pressure cylinder are placed side by side, each actuating its own piston rod and each piston rod direct connected to its own water cylinder. This type is known as the "cross-compound," and has been developed to a high state of efficiency, probably surpassing any other type for capacities up to about 10,000,000 gallons per day.

The advantages of two or three water cylinders discharging into the same discharge pipe have already been explained. To secure this advantage, many water works pumps, and all of the high-duty ones, consist of practically two or three engines built side by side as part of the same machine. The cross-compound referred to is a two-cylinder or duplex engine, and most of the high-duty water-works engines have been duplex. The triple-

expansion engine arranged as a direct-connected engine introduced a number of objectionable features, one of these being the great length of engine and foundation with the three cylinders placed in line, another being the difficulty of getting at the middle of the three cylinders for repairs of any kind. The engine was expensive and this type was ordinarily used for only engines of more than six or eight million gallons per day. In the water end of a horizontal engine the suction is measured to the valve deck, which is near the top of the water cylinder, and as every foot of suction lift is an important matter, the additional suction lift above engine foundation level caused by the large water cylinders was a disadvantage. Mechanical difficulties connected with securing a sufficiently strong and inflexible structure on a single engine bed was another objectionable feature of horizontal triple-expansion engines of large capacity.

Several of these objections are overcome by the vertical engine, in which the water cylinder is placed at the floor level and the steam cylinder vertically above it, either direct connected or with crank and flywheel. The most efficient form of high-duty vertical engine to date is a triple expansion one in which the three steam cylinders are placed side by side, as in the cross-compound, all three connected to and driving a crank shaft which carries two flywheels, and also direct connected to the plungers of three water cylinders. In these engines the pump ends are made single acting, with outside-packed plungers; and as the water is all taken in at the bottom of the cylinder, the suction lift is not increased by increasing the size of the pump. Another advantage in some localities is that, in case of flooding of the pumping station, the water can rise several feet above the floor without affecting the pumping engine, since the steam cylinders are the only portions which would be affected by water.

Reviewing the above as to type and form of engine, we find in water-works service (although not all of them are now being manufactured) compound, non-condensing, horizontal; compound, condensing, horizontal; triple, non-condensing, horizontal; triple, condensing, horizontal; each of the latter two either

with or without Worthington high-duty attachments, and all four of the direct-acting type. Of the crank and flywheel or rotative type we find the cross-compound, condensing, high-duty, both horizontal and vertical; a double compound, condensing, high-duty, horizontal (the Gaskill-Holly); the triple, condensing, high-duty, horizontal; and the triple, condensing, high-duty, vertical. The horizontal, compound, non-condensing, direct-acting engine is not adapted to pumping capacities of more than about 1,000,000 gallons per day, and even for engines of this capacity is uneconomical except where coal is very cheap. The horizontal, triple, condensing, direct-acting, known as the low-duty triple, gives good service up to 6,000,000 gallons capacity, and is a good compromise between a low-duty and high-duty pump for engines up to this size. The high-duty, triple, condensing, direct-acting engine, either horizontal or vertical, is the most economical direct-acting engine in service and is used up to 40,000,000 gallons per day. Of the crank and flywheel engines, the horizontal Holly was an excellent engine and many are still in service giving quite high duty. The old Holly, however, was very extravagant of space, and higher duties are now obtainable at a lower cost by means of the Reynolds type of vertical, triple-expansion, condensing, crank and flywheel engine, which is probably the best obtainable for capacities of over 10,000,000 gallons per day. For capacities below this the horizontal cross-compound has for a number of years been generally favored. The above gives the broad lines by which different pumping engines are distinguished, but there are numerous minor details which enter into the designing and construction of a pump which aid in securing high efficiency, low maintenance cost, and certainty of continuous operation. The last is by no means the least important requirement, since a pump which frequently breaks down is not only expensive for repairs and annoying, but may cause enormous fire losses through failure to operate at a critical time. The details as to size, type and material employed in the valves of both steam and water ends, the construction of stuffing boxes and other details cannot be considered here. The chief features desirable in the pump,

with or without Worthington high-duty attachments, and all four of the direct-acting type. Of the crank and flywheel or rotative type we find the cross-compound, condensing, high-duty, both horizontal and vertical; a double compound, condensing, high-duty, horizontal (the Gaskill-Holly); the triple, condensing, high-duty, horizontal; and the triple, condensing, high-duty, vertical. The horizontal, compound, non-condensing, direct-acting engine is not adapted to pumping capacities of more than about 1,000,000 gallons per day, and even for engines of this capacity is uneconomical except where coal is very cheap. The horizontal, triple, condensing, direct-acting, known as the low-duty triple, gives good service up to 6,000,000 gallons capacity, and is a good compromise between a low-duty and high-duty pump for engines up to this size. The high-duty, triple, condensing, direct-acting engine, either horizontal or vertical, is the most economical direct-acting engine in service and is used up to 40,000,000 gallons per day. Of the crank and flywheel engines, the horizontal Holly was an excellent engine and many are still in service giving quite high duty. The old Holly, however, was very extravagant of space, and higher duties are now obtainable at a lower cost by means of the Reynolds type of vertical, triple-expansion, condensing, crank and flywheel engine, which is probably the best obtainable for capacities of over 10,000,000 gallons per day. For capacities below this the horizontal cross-compound has for a number of years been generally favored. The above gives the broad lines by which different pumping engines are distinguished, but there are numerous minor details which enter into the designing and construction of a pump which aid in securing high efficiency, low maintenance cost, and certainty of continuous operation. The last is by no means the least important requirement, since a pump which frequently breaks down is not only expensive for repairs and annoying, but may cause enormous fire losses through failure to operate at a critical time. The details as to size, type and material employed in the valves of both steam and water ends, the construction of stuffing boxes and other details cannot be considered here. The chief features desirable in the pump,

to which all of these details contribute, are economy in the use of steam and reliability of operation. For the latter, reliance is generally placed upon the reputation of the manufacturer and a guarantee that, during several months of operation, there shall be no undue wear or breakage due to fault in design, workmanship, or materials used in the pump. Steam economy will be considered under the head of duty, in Art. 61.

Art. 61. Duty of Pumping Plants

The duty of a pumping plant is a manner of expressing its efficiency—the amount of work done in pumping water per unit of energy employed. The work done is the product of the pounds of water lifted and the feet head (or 2.306 times the pounds pressure) against which the pump operates. The energy is the heat used, and may be expressed in terms of the amount of steam, coal, gas, or oil supplying the heat, or of the heat itself expressed in British thermal units (B.t.u.).

In the majority of cases pumps or boilers, or even the entire plant, are sold with a guarantee that they will operate with a certain named efficiency. This is the maximum efficiency that they will develop at a special test, and it should not be assumed that it will be maintained during service operation; for cheaper fuel may be used than the selected coal used in the test, the firemen be less experienced or not always so alert, the pump be not always in perfect adjustment, the rate of pumping or head be not often that for which the pump is most efficient, and the pumps shut down and the fires inoperative at night. J. N. Chester considers that pumps that developed 50 million and 200 million duty respectively per 1000 pounds of steam should not be expected to show a service duty, figured on all the coal used under the boilers, of more than 20 million and 125 million respectively per 100 pounds of coal.

The energy applied may be taken at the throttle valve of a steam pumping engine, or at the shaft of a power pump, in which case we have the duty of the pumping engine or pump alone. Or it may be taken as the amount of coal or of heat

in the same used in furnishing steam to the pumps; or as the total amount of coal used for this purpose and also for lighting the plant and all other purposes connected with it. The last, called station duty, is not strictly duty, but is a factor in determining the total cost of operating the pumping plant.

PUMPING ENGINES

Duty of a pumping engine alone is expressed as foot-pounds per 1000 pounds of dry steam used, or per 1,000,000 British thermal units. These two are approximately equal, but not exactly; that is, 1 pound of steam furnishes about 1000 heat units at 75 pounds pressure, but more at higher pressure. (A heat or British thermal unit is the amount of heat necessary to raise 1 pound of water at its maximum density through 1° F.) As an example, in a test of a 16,000,000-gallon pumping engine it was found that the duty on the basis of million heat-units was 136,260,000 foot-pounds, on the basis of 1000 pounds of moist steam, 146,459,000; and on the basis of 1000 pounds of dry steam, 147,269,000. The B.t.u. (British thermal units) in a pound of dry steam is represented by the equation $H=1150.4+0.35(T-212)-0.000333(T-212)^2$, in which T is the temperature Fahrenheit. The B.t.u. in water are 180.0 at 212°—that is, just before turning to steam at atmospheric pressure; the difference, or 970.4 B.t.u., being latent heat. The B.t.u. in water at the boiling-point increase with the pressure, being 391.3 at 300°, and the latent heat of evaporation at the same point is 817.0, the total heat content being 1208.3. If steam enters a steam cylinder at 300° and leaves it as water at 212°, it has made available in the cylinder 1028.3 B.t.u.

Water turns to steam at 212° at atmospheric pressure, but the steaming temperature increases with the pressure, being 327.9° at 100 pounds, and 417.4° at 300 pounds. As long as the steam remains in contact with the water it cannot be raised above its steaming or vaporizing temperature; but if removed from the boiler and passed through heating coils (called a "superheater") on its way to the engine it can be heated still higher, the additional heat being called "superheat." If saturated steam (that

is, not superheated) is cooled by any amount, part of it turns to water; but superheated steam can lose all the superheat before condensing. Dryness of steam on entering the steam cylinder is exceedingly desirable.

In testing a pumping engine on the basis of pounds of dry saturated steam, the amount of such steam used during the test would be ascertained by weighing all the water fed to the boiler (including any needed at the end of the test to fill the boiler to just the amount at which it started), and deducting from this any leakage from pipes (the loss from steam pipes and boiler is seldom less than 0.5 per cent, may be 2 per cent in a test, and 10 per cent is not uncommon in service), calorimeter waste, and the amount of entrained water passing the throttle valve (which is determined by frequent tests of water blown from the calorimeter outlet). To this will be added an allowance for any superheat, the superheat being reduced to pounds of steam that could be made by the superheat units.

If the duty is on the million heat units basis, the heat units figured are those brought by steam to the main engine and its appurtenances and auxiliaries, these including steam cylinders, steam jackets, reheating coils, steam feed pump, and air pump (where these are not operated by the main engine, as is frequently the case). In the calculation, allowance must be made for entrained water in the steam and for superheat. If there is x per cent of entrained water or moisture in the steam when it reaches the engine, the heat units obtained by taking the amount of steam supplied and the sensible temperature of such steam must be diminished by x per cent of the latent heat at the pressure recorded. The superheat units per pound of steam are found by multiplying by 0.48 the difference between the temperature of the superheated steam and that of saturated steam at the indicated pressure. The engine is credited with heat in the water returned to the boiler from the condenser, exhausts, jackets, etc., determined by the amount and temperature of water from each.

The work done is determined by measuring the amount of water pumped, and the pressure head against which it is

pumped, and the suction head, both heads measured from a common datum. The quantity can be measured most accurately by discharging into a standpipe, the depth of water in which is determined at the beginning and end of the test by measuring down to the surface from the top of the tank with a steel tape; the pump discharge being throttled at the beginning and gradually opened up to keep the pressure constant as the water rises in the standpipe. If a steel tank is not available of sufficient size for the entire test, it can be used to determine exactly the amount discharged by a given number of strokes of the pump, such determination being made at the beginning and end of the test, and at intervals during it if it lasts more than twelve hours; and with this as a basis and a pump counter to register the number of engine strokes, the total amount pumped can be calculated quite exactly if the pump stroke is of unvariable length, as is the case with crank-and-flywheel pumps. Other methods employed are by use of weirs, venturi meters, nozzles of known delivery under certain heads, pitometer measurements, etc., any of which are subject to an error of 1 per cent, and of much more if not handled by experts. Or the amount may be determined by calculation from the diameter of plunger and length of stroke, both very carefully measured, with an estimated allowance for slip of 1 or 2 per cent, the disadvantage of the latter being the uncertainty as to the amount of slip.

The pressure head may be measured by a pressure gage, tested for accuracy just before the test, or by a mercury column connected to the discharge, the latter being the more reliable. The suction head may be measured by a vacuum gage; or, if there is a suction well beneath the pump, by measuring the depth of water in this below the pressure-head zero.

The product of total water pumped, times its weight per unit, times the sum of suction and pressure heads, gives the foot-pounds of work done; and this divided by $\frac{1}{1000}$ of the weight of dry steam, or by $\frac{1}{1.000.000}$ of the B.t.u., used by the engine during this time, gives the duty. The highest grade steam engines seldom deliver as work more than 20 per cent of the energy received as heat units, and many small ones do not show

more than 5 per cent efficiency. Engines of more than 1,000,000 gallons per twenty-four hours capacity have developed duties of from 50 million to 200 million foot-pounds per 1000 pounds of steam, the last only by large triple crank-and-flywheel engines; while small, cheap engines may fall as low as 10 million.

BOILER EFFICIENCY

The efficiency of the boiler that generates the steam is of nearly as great importance as that of the pumping engine; and a high efficiency of the entire plant—boiler, steam-pipes, engine and pump—acting together is the end sought. In many cases this total efficiency is guaranteed by one firm, which furnishes the entire plant in running order; and this efficiency is then determined, expressed as foot-pounds of work per 100 pounds of coal or of combustible matter in the coal, or per 1,000,000 heat units in the coal. The efficiency of the boiler alone is expressed in the number of pounds of steam formed, "from and at 212°," for each pound of coal or combustible matter. To determine the combustible matter, the weight of ashes is deducted from the weight of coal burned. The ashes may be between 2.5 per cent and 25 per cent or more, some grates giving 5 per cent or more of unburned combustible matter in the ashes. Different coals vary 30 per cent or more in their available heat units, or from 15,000 B.t.u. and over to 11,000 and less per pound; and therefore the heat unit basis is much more definite than the pounds of coal.

The number of pounds of water that can be evaporated by a given number of heat units depends upon the original temperature of the water, and that of the steam at evaporation, which latter varies with the boiler pressure. Table No. 39 shows the boiling or evaporating point for various steam pressures in the boiler, these being the absolute pressures, which are 14.7 pounds more than those indicated by the steam gage.

The higher the evaporating point, the greater the amount of heat in saturated steam. Taking the heat required to evaporate water from and at 212° (that is, at 14.7 pounds pressure) as 1.00 that at 428° is seen by the table to be 1.07. It

takes a certain amount of heat to raise the temperature of water each degree, and the table shows that it takes approximately 1 per cent as much to raise the temperature of water 9° as to change it to steam at 212°. Hence the number of pounds of water which can be evaporated by a given amount of heat depends upon the original temperature of the feed water and the boiler pressure. In order to compare different tests, the actual duty performed by coal in a given test is reduced to that it would have performed had the feed water been at a temperature of 212° F. when reaching the boiler, and the steam gage registered zero. If the temperature of the water be lower than 212°, or the absolute pressure more than 14.7, more heat units will be required. As an illustration of the use of Table No. 39: if 1 pound of coal evaporate 8 pounds of water whose original temperature was 50°, the steam pressure being 182.4 pounds per square inch, the work done was equivalent to evaporating 1.22 times 8 pounds or 9.76 pounds, from and at 212°.

TABLE NO. 39

HEAT CONTENT OF SATURATED STEAM AT DIFFERENT PRESSURES

Absolute Pressure per Sq. In.	Boiling-point.	Initial Temperature of Feed Water.										
		32°	50°	68°	86°	104°	122°	140°	158°	176°	194°	212°
14.7	212°	1.19	1.17	1.15	1.13	1.11	1.10	1.08	1.06	1.04	1.02	1.00
20.8	230	1.20	1.18	1.16	1.14	1.12	1.10	1.08	1.06	1.04	1.02	1.01
28.83	248	1.20	1.18	1.16	1.14	1.13	1.11	1.09	1.07	1.05	1.03	1.01
39.25	266	1.21	1.19	1.17	1.15	1.13	1.11	1.09	1.07	1.06	1.04	1.02
52.52	284	1.21	1.20	1.18	1.16	1.14	1.12	1.10	1.08	1.06	1.04	1.02
69.21	302	1.22	1.20	1.18	1.16	1.14	1.12	1.11	1.09	1.07	1.05	1.03
89.86	320	1.22	1.21	1.19	1.17	1.15	1.13	1.11	1.09	1.07	1.05	1.03
115.1	338	1.23	1.21	1.19	1.17	1.15	1.14	1.12	1.10	1.08	1.06	1.04
145.8	356	1.23	1.22	1.20	1.18	1.16	1.14	1.12	1.10	1.08	1.06	1.04
182.4	374	1.24	1.22	1.20	1.18	1.17	1.15	1.13	1.11	1.09	1.07	1.05
225.9	392	1.24	1.23	1.21	1.19	1.17	1.15	1.13	1.11	1.09	1.07	1.06
276.9	410	1.25	1.23	1.22	1.20	1.18	1.16	1.14	1.12	1.10	1.08	1.06
336.3	428	1.26	1.24	1.22	1.20	1.18	1.16	1.14	1.12	1.11	1.09	1.07

The efficiency of a boiler is the quotient obtained by dividing the added heat units in the steam that leaves the boiler within a given time by the heat units furnished by the coal or other fuel.

Few boilers show an efficiency exceeding 86 per cent, and in service many fall as low as 60 per cent.

From Grate to Pump Discharge. To understand and study the energy furnished, utilized, and lost in a steam plant, it should be borne in mind that steam is but an agent for utilizing heat, and that all the heat furnished by the fire can and should be accounted for in a test. The following details of a test of a compound engine are here given, accompanied by a graphic representation of the same facts, prepared from the data by

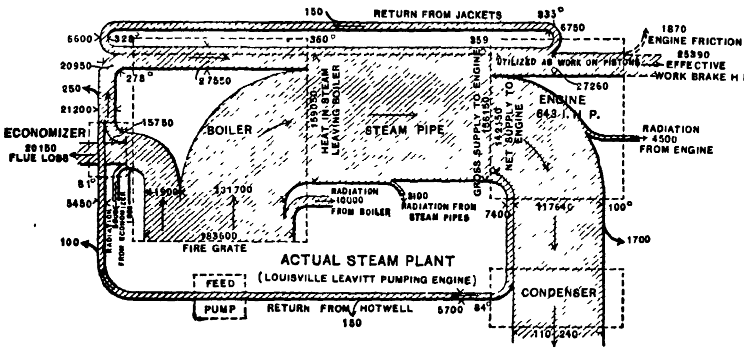


FIG. 62.—Loss of Heat in a Steam Pumping Plant.

Capt. H. R. Sankey and presented by him in a report to the Institution of Civil Engineers. It is thought that a careful study of these, and particularly of the diagram, will be of great value in getting clearly in mind the losses in, and efficiency of, the various parts of a steam pumping-plant.

"Starting at the fire grate, it is shown that 183,600 B.t.u. are produced per minute by the combustion of the coal, and that 131,700 of these go direct into the water of the boiler, 10,000 are lost by boiler radiation and leakage, and the remainder, viz., 41,900, pass away with the flue gases. On the way to the economizer 1000 B.t.u. are lost by radiation, but in the economizer itself 15,750 B.t.u. are diverted into the feed water, 5000 B.t.u. are dissipated by radiation, and finally 20,150 B.t.u. pass out of the economizer and into the chimney, and are lost to the steam plant. The heat entering the economizer with the feed water is 5450 B.t.u., which is added to the 15,750 B.t.u. diverted from the flue gases, thus giving a flow of 21,200 B.t.u. in the feed out of the economizer. Radiation, however, reduces this flow to 20,950 B.t.u. per minute at the entry to the boiler, where a further addition is made of 6600 B.t.u. returned by the jacket water.

"The steam produced by the boiler is thus seen to derive its heat from three streams, as shown in the diagram; the steam finally leaves the boiler with 159,250 B.t.u. per minute. Before this heat gets to the engine, however, 3100 B.t.u. are lost by radiation and leakage from the steam pipes, so that the flow of heat

is reduced to 156,150 B.t.u. per minute, which is the gross supply of heat to the engine; the net supply is less, because there are certain returns of heat to the boiler to be deducted. In the first place, credit has to be given to the engine for the heat which could be imparted by means of the exhaust steam to the feed water, inasmuch as the exhaust is theoretically, and very nearly practically, capable of raising the temperature of the feed to the exhaust temperature. On this basis, 7400 B.t.u. should be credited to the engine. Although the actual return to the boiler, or rather to the economizer, is only 5450 B.t.u."

Of the 6750 "return from jackets," the 150 lost by radiation should be charged against the engine, since the jackets are supplied to correct a fault in the engine, viz., cylinder condensation. This leaves 142,150 B.t.u. as the net supply to the engine. Of this, 117,640 departs in the exhaust without yielding any of its energy, and the remainder, or 27,260 B.t.u., goes into work in the engine. Of this, 1870 is consumed in the internal friction, leaving the actual work done in pumping due to 25,390 B.t.u. The thermal efficiency of any part of the plant can now be readily determined, and is the ratio of the number of B.t.u. applied to those delivered as heat or work. Thus that of the engine is $\frac{25,390}{142,150} = .18-$, the per cent of applied energy utilized. The thermal efficiency of the boiler, economizer, and grate is $\frac{159,250}{183,600+5450+6600} = .81\frac{1}{2}$; and of the entire plant $\frac{25,390}{183,600} = .14-$. The mechanical efficiency of the pump is $\frac{25,390}{27,260}$, or $.93+$.

The following are some of the data obtained in the test of this engine, the table also serving to illustrate the kinds of information obtained in a duty test.

STEAM USED BY ENGINE AND FEED PUMP; ENTIRE TEST OF 144 HRS. 10 MIN.

(1)	Weighed feed water	968,128 lbs.
(2)	Feed-pump steam, condensed	23,390 lbs.
(3)	Total water pumped into boilers	991,518 lbs.
(4)	Total water returned to boilers from jackets and reheaters	189,795 lbs.
(5)	Sum of (3) and (4)	1,181,313 lbs.
(6)	Total steam used by calorimeter	727 lbs.
(7)	Total water drained from separator	23,428 lbs.
(8)	Total moist steam used by engine and feed pump	1,157,158 lbs.
(9)	Percentage of moisture in steam after leaving separator	0.55%
(10)	Total dry steam used in engine and feed pump (=99.45% of (8))	1,150,792 lbs.

STEAM USED BY ENGINE

(11)	Total moist steam used by engine only	1,133,768 lbs.
(12)	Total dry steam used by engine only	1,127,533 lbs.
(13)	Total moist steam passing through cylinders	943,973 lbs.
(14)	Total moist steam passing through jackets and reheaters	189,795 lbs.
(15)	Percentage of moist steam used by jackets and reheaters	16.74
(16)	Moist steam used per hour per i h.p. (indicated horsepower) *	12.223 lbs.
(17)	Dry steam used per hour per i.h.p.*	12.156 lbs.
(18)	Dry steam passing through cylinders per hour per i.h.p.	10.120 lbs.
(19)	Dry steam used per hour per pump h.p.*	13.050 lbs.

B.T.U. SUPPLIED BY BOILERS

(20)	Heat of vaporization, steam 154.6 lbs., absolute †.....	859.4 B.t.u.
(21)	Heat of liquid, steam 154.6 lbs.†..................	332.5 B.t.u.
(22)	Heat of liquid feed, 143.30.......................	111.5 B.t.u.
(23)	Per pound of moist steam supplied by boilers (859.4×.9945+332.5−111.5=)........................	1,075.7 B.t.u.
(24)	Total supplied by boilers, ((5)) 1,181,313×1075.7=.......................	1,270,738,600 B.t.u.
(25)	Total supplied by boilers per minute................	146,903 B.t.u.

B.T.U. USED BY THE ENGINE

(26)	Per pound of moist steam used by the cylinders.......	1134.5 B.t.u.
(27)	Per pound of moist steam used in jackets and reheaters	880.8 B.t.u.
(28)	Used by engine during trial..........................	1,238,108,959* B.t.u.
(29)	Used by engine per minute........................	143,134 B.t.u.
(30)	Used by engine per minute per i.h.p................	222.46 B.t.u.
(31)	Average mean effective pressure in high-pressure cylinder..................................	43.53 lbs.
(32)	Average mean effective pressure in low-pressure cylinder..	14.155 lbs.
(33)	H.p. developed in H.P. cylinder.....................	279.00
(34)	H.p. developed in L.P. cylinder....................	364,40
(35)	H.p. lost in friction..............................	44.30
(36)	Efficiency of mechanism....	93.12%

* I.h.p., the energy utilized as work on the pistons, represented in the diagram by 27,260 B.t.u. Pump h.p., the energy delivered by the pump in work, represented as 25,390 B.t.u. per minute.

† Note that the sum of (20) and (21), or total heat units, gives the same result as the formula for H; the temperature of the steam being 365°.

FUEL

		Coal. Pittsburgh.	Coal. Pocahontas.
(37)	Moist coal consumed, lbs..........................	67,917	63,591
(38)	Wood consumed at 50 per cent weight, lbs...........	772	25
(39)	Moisture in coal, per cent........................	0.7	2.6
(40)	Dry coal consumed with wood equivalent, lbs.........	67,995	61,692
(41)	Total ash, dry, lbs................................	2,025	2,849
(42)	Total combustible, lbs.............................	65,970	58,843
(43)	Calorific value of one pound of coal by analysis, B t.u..	13,226	14,924
(44)	Water actually evaporated per pound of dry coal, lbs..	8.60	9.57
(45)	Equivalent water evaporated per pound of dry coal, from and at 212°, lbs..............................	9.63	10.70
(46)	Equivalent total heat derived from a pound of dry coal, B.t.u..................................	9,250	10,294
(47)	Water actually evaporated per pound of combustible, lbs..	8.86	10.03
(48)	Equivalent water evaporated per pound of combustible, from and at 212°, lbs..............................	9 92	11.22
(49)	Efficiency of boilers, per cent $\left(=\frac{(46)}{(43)}\right)$..............	70.0	69.0
(50)	Dry coal burned per sq. ft. of grate per hour, lbs......	12.70	11.50
(51)	Coal used per i.h.p. per hour, lbs...................	1.47	1.33
(52)	Coal used per pump h.p., per hour, lbs..............	1.58	1.43

POWER PUMPS

The efficiency of power pumps is not generally expressed as duty, except in the case of direct or gear-connected internal-combustion engines, when it may be calculated as the duty of both engine and pump per 1,000,000 heat units in the gas, gasoline, or oil used. In that case the work is ascertained as for steam-pumping engine duty and compared with the heat units in the gas or oil used as determined by analysis. Or the efficiency of the pump alone may be expressed as the percentage of applied work that is utilized in pumping water, the latter being determined as before and the applied energy being measured by a transmission dynamometer. If the pump is driven by an electric motor, the efficiency of the combination may be stated as footpounds of work done per kilowatt-hour.

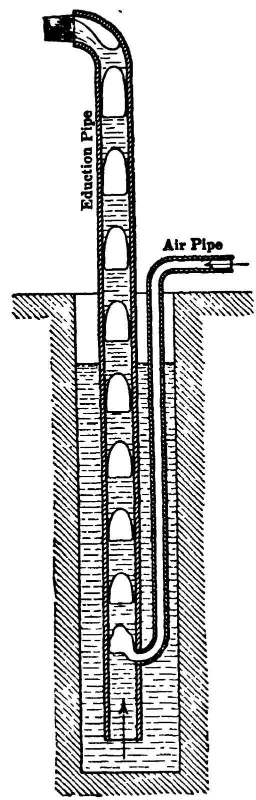


FIG. 63.—Pohle System of Operation.

In the case of centrifugal pumps the efficiency of the pump alone may be expressed in any of the above ways; or, if driven by a steam turbine, as duty in terms of steam or heat units used by the turbine. A steam-turbine-centrifugal-pump unit may develop 70 to 150 million foot-pounds per 1000 pounds of steam. The amount of steam used is that entering the nozzles of the pump, plus that used in air (vacuum) and condenser pumps if these are not driven from the main pump shaft, which they generally are.

ART. 62. THE AIR LIFT

What is known as the air lift was first patented in 1884, but little use was made of it before 1892, following the issuing of a

patent therefor to J. G. Pohle. The process consists in raising water by mixing air with it in a vertical tube, the effect being to produce a mixture lighter than water, which consequently is raised in the tube by the hydrostatic pressure of water at the lower end of the tube.

According to the principles of hydrostatics, the vertical pressure per square inch of the liquid in the tube exerted at the base thereof, will equal the pressure exerted from without at the same point if there is no motion; and if the pressure from without

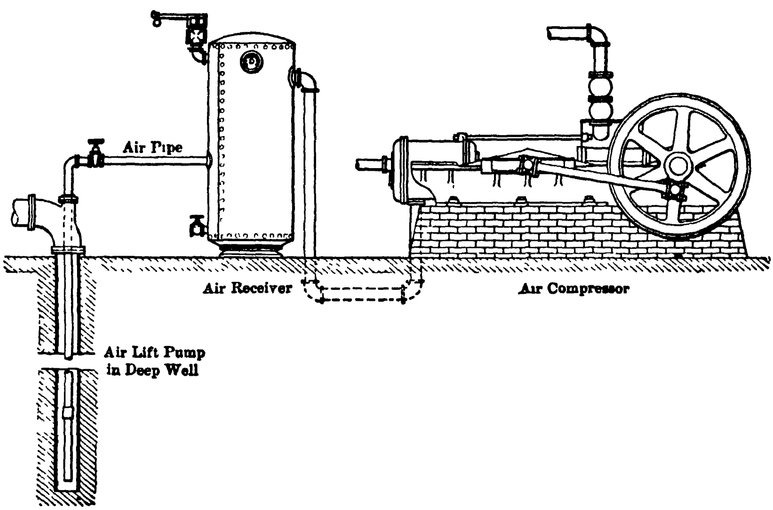


FIG. 64.—Air Lift Pumping Plant.
(From Bulletin No. 450, University of Wisconsin.)

exceeds that due to the weight of the liquid in the tube, the latter will move upward. For instance, if the bottom of a tube well is lowered to a depth x feet below the surface of the ground water, and if the mixture of air and water in the tube has a specific gravity of 0.5, the mixture in the well will rise $2x$ feet above the bottom; and if the top of the well be cut off at any point less than $2x$ feet above the bottom, water will overflow at the top, and this overflow will continue so long as the air and water mixture is maintained at a specific gravity of .5. The specific gravity, of course, depends upon the relative amount

of air added to the water in the tube. It is found by experience that it is not economically practicable, in lifting water, to reduce the specific gravity much below .5 by the introduction of air, effort to do so resulting in a waste of air.

Since the air must be blown into the water near the bottom of the well, pressure must be exerted to force the entrance of air against the hydrostatic head of the water. This is effected by use of an air compressor, the air being carried down to a point near the bottom of the well by means of an air pipe. The apparatus employed in the air-lift pump, therefore, consists of a tube well, an air compressor, an air pipe for introducing the air, and some form of construction at the point where the air enters the well. The well or pipe through which the water is discharged is known as the eduction pipe, lift tube, etc. The pipe introducing the air is known as the air pipe; the special construction at the juncture of the two is called the foot piece. In most constructions the well casing is extended below the foot piece for a short distance to prevent the escape of the compressed air from the bottom of the pipe, such extension being known as the tail piece. The machinery for furnishing the air consists of an air compressor of any type and an air receiver for storing and equalizing the air pressure. The receiver also acts as a separator to catch the water and oil which may be carried by the air from the compressor. The air from the compressor passes down through a pipe which enters at the top of the receiver and discharges beneath the surface of water which stands in the bottom of the receiver. The outlet pipe is located some distance up from the bottom of the receiver. The air pipe from the receiver to the wells should be made as direct as possible, without any unnecessary valves, elbows or joints, as these reduce the efficiency. The pipe also should be made air tight. Both a regulating and a stop valve should be placed on the air branch connecting the air main with each well, so that the flow of air to each individual well can be adjusted; and once having been adjusted, the supply can be shut off or turned on without altering the adjustment.

The construction of the eduction pipe and air tube differs

in different systems. In some cases two separate pipes serving these purposes are lowered into the well-casing; in another construction, an eduction pipe is lowered inside the casing and the air introduced through the annular space between eduction pipe and well-casing; and in still another construction the casing serves as the eduction pipe and the air is introduced through a small air tube which is placed in the center of the well-casing. These several forms are shown in Fig. 65.

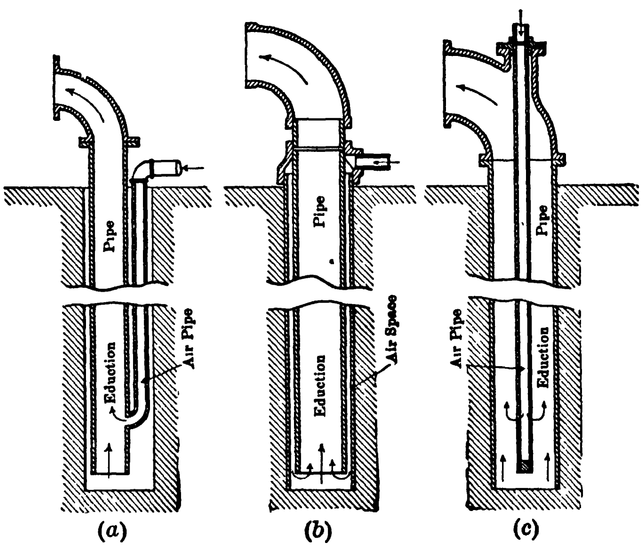


FIG. 65.—Different Methods of Piping Wells.

A number of different and more or less intricate constructions have been devised for the foot piece, but it is believed by some investigators of the subject that these have little beneficial effect and may have a detrimental effect if they contract the water way and thus increase the friction of the water entering the eduction pipe; and that the most satisfactory as well as simplest plan is to simply admit the water through a plain opening either in the side of the eduction pipe or, in the case of the center air-tube construction, at or near the bottom of the air tube. Mathematical calculation of the theoretical discharge that can be obtained with a given size of well-casing and using the three

classes of construction mentioned gives the following relative quantities: side inlet, 1.00; annular air tube, 1.81; central air-tube construction, 2.84. These calculations were made on the assumption of a 6-inch well, a 1-inch air tube in the case of the side inlet, a $1\frac{1}{2}$-inch air tube in the case of the central air tube, and a $4\frac{1}{2}$-inch eduction pipe in the case of the annular tube system.

The exact action of the air in the eduction pipe is difficult to determine, but it is believed that it rises in the form of large bubbles, each nearly filling the pipe, rather than in a great number of small bubbles. No perfectly satisfactory theory has been evolved by which to calculate the operation or efficiency of the air lift, but its development has been based almost exclusively upon the results of experiments and experience with operating installations. From these it appears that the submergence (that is, the depth from the surface of the ground water to the point where the air is admitted) should be between 60 and 65 per cent of the total length of the air lift, measured from the air inlet to the point of discharge. This means that the depth of the well and its casing should be about three times the lift from the ground water level to the point of discharge. (The elevation of ground water in making these measurements is that at which such water stands in the well under working conditions rather than under static conditions, which is several feet lower than the normal ground-water elevation.) It follows from this that each foot by which the point of discharge of the well is lowered reduces the required depth of the well by about 3 feet. It also emphasizes the desirability of repumping the discharge where the water is to be raised to a tank or reservoir, rather than attempting to use the air lift for the complete elevation of the water to such structures.

Various tests made seem to indicate that the following laws are generally applicable. For a given size of eduction pipe and a constant ratio of submergence to lift, the rate of delivery of water and the air consumption per gallon are practically constant; although, of course, the power required to introduce the air varies with the depth of submergence. If both sub-

mergence and lift remain constant, there is a definite quantity of air which causes the maximum discharge, the amount of discharge diminishing if the amount of air is either increased or decreased. The efficiency of the energy applied in the form of compressed air, however, increases as the rate of pumping decreases, with a given size of eduction pipe. For a constant lift, the efficiency increases as the submergence increases. For a given length of well with varying percentages of submergence, the maximum efficiency is obtained when the submergence is approximately 63 per cent (according to the experiments of the University of Wisconsin). With a given lift and submergence, the air consumption per gallon decreases as the size of the discharge pipe increases. The least air pressure that will give continuous flow is the proper one to use. If the pressure is lowered slightly, the delivery becomes intermittent and the amount delivered is considerably decreased. With pressure higher than that which is just sufficient to give continuous flow, the delivery is increased somewhat, but the air consumption per gallon is increased in a still higher ratio; and if the air pressure continue to be increased, a point of maximum delivery is finally reached beyond which the amount of delivery actually decreases with an increase in air pressure. After an operator has become familiar with a system, he can use the sound of the discharge as a reliable guide for a proper regulation of the air supply.

One conclusion from the above is that, with a given depth of ground water, increasing the depth of well reduces the air consumption, but at the same time increase the air pressure so that a cubic foot of air delivered represents more power exerted by the air compressor. However, the total power required per gallon of water delivered at first decreases as the depth increases, but finally reaches a maximum beyond which the increase in pressure more than offsets the decrease in quantity, and the total power required per gallon of water delivered is increased. For a given lift, the depth where this maximum efficiency of power is obtained decreases as the size of the discharge pipe increases.

A duty test made of a large plant in Atlantic City, N. J.,

showed a duty of between 20,000,000 and 25,000,000 foot-pounds of work per thousand pounds of dry steam, the pumping plant consisting of a Rand duplex flywheel compressor, with 13- by 12-inch air cylinder. D. W. Mead has estimated that the duty in million foot-pounds per thousand pounds of dry steam would probably vary from 19 to 30 with a compound Corliss compressor, to from $4\frac{1}{2}$ to 10 with a small straight-line compressor. A comparison between air-lift and steam-operated deep-well pumps at Waukesha, Wis., showed an efficiency of between 16 and 18 per cent for the air lift, based on the indi-

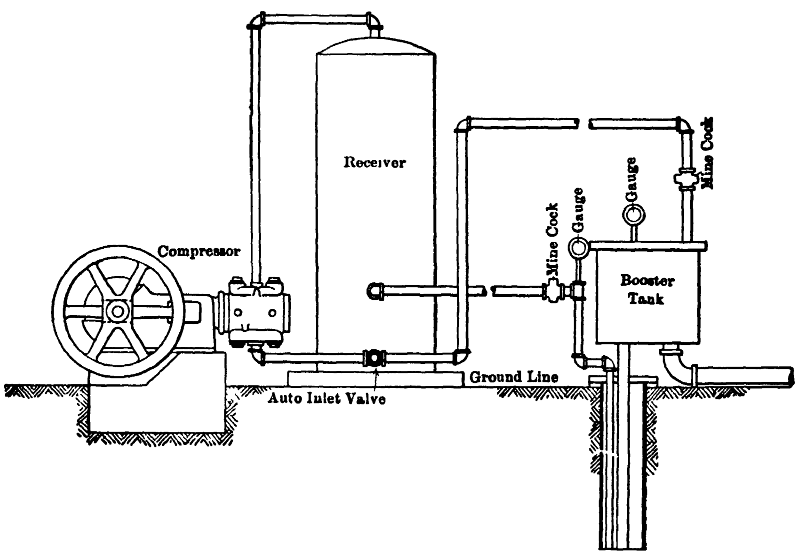


FIG. 66.—Layout for " Booster " System of Air Lift.

cated horse-power in the steam cylinder of the engine, and an efficiency of 74.8 per cent for deep-well pumps on the same basis.

As the air bubbles rise through the well, the pressure upon them exerted by the water decreases and the bubble accordingly expands. It still, however, retains a certain amount of internal pressure, especially if the water after leaving the well is to be forced through any considerable length of horizontal pipe. Recently a device known as a " booster " has been perfected for utilizing the remaining pressure in the bubbles, the

air being collected in the booster and its remaining expansive force being utilized to give additional pressure to the water in the delivery pipe leading from the well. In addition, the air in the booster is further compressesd by the kinetic energy of the water which leaves the well with considerable velocity. By use of the booster, air-lift equipments have been operated for raising the discharged water through an additional head to elevated tanks or other receptacles. The air from the booster is sometimes returned to the air compressor because it is colder than outside air, and by every 5° lowering of the air temperature a volumetric gain of 1 per cent is secured.

The advantages and disadvantages of the air lift have been summarized as follows by George J. Davis, Jr., and C. R. Weidner, of the faculty of the University of Wisconsin (Bulletin No. 450 of the University of Wisconsin):

Advantages: The principal advantage of the air lift is its large capacity. When conditions are suitable for its installation, an air-lift pump will discharge more liquid from a well of small bore than will any other type; this being chiefly because almost the entire cross-sectional area of the well is available for the flow of the water and the action is nearly continuous. The quantity that can be discharged by an air lift is limited only by the capacity of the well and the expense of pumping; while with deep-well pumps, the majority of which are single acting, the discharge is limited by the allowable piston speed, which usually does not exceed 100 feet per minute. (This limitation does not apply to centrifugal deep-well pumps, but these are affected by other limitations occasioned by their method of operation.) The air lift affords a desirable means of testing the capacity of a well, even if it is not proposed to install it permanently.

The maintenance cost of an air-lift plant is very low, owing to its simplicity. The operation is exceedingly simple and the life of the equipment is almost indefinite. Should the air pipes and foot piece become clogged with oil carried over from the compressor cylinders, they must be removed and cleaned; but this rarely occurs, and when it does, the cost is small compared with that of replacing or repairing a mechanical pump.

Where conditions are such that the wells are scattered over a considerable area or are remote from the power-house, the air lift has a decided advantage over the steam-driven pump in that, if the latter is used, each well must be equipped with a separate engine and working barrel, which entails heavy condensation losses through long steam supply pipes, as well as considerable cost of equipment. Also the expense of attendance of a plant of this sort, with its scattered pumping units, is great. In the air-lift pump the transmission loss is much smaller, and as no attendance at the wells is required, they may be put into operation or controlled by a valve in the power-house.

Where water is to be used for a domestic supply and there are impurities in it such as iron, it has been noticed that the iron is oxidized by the aeration of the water and the supply is thereby improved.

As to reliability, air-lift pumps are not liable to sudden stoppages or breakdowns.

Disadvantages. In common with every other device, the air-lift pump has certain disadvantages, the principal ones being its low efficiency, the great depth of well required, and its poor adaptability to conditions requiring a discharge to be conveyed over great horizontal distances. The most serious of these is the low hydraulic efficiency. This is usually given as 25 to 33 per cent. Notwithstanding the low efficiency of the pump itself, however, the entire air-lift plant in some cases develops a duty which compares favorably with other systems of pumping.

A single air-lift pump cannot be used in a shallow well or reservoir except to raise the liquid a very small distance, owing to the high percentage of the total length of the pump that must be submerged. This limits the field of air-lift pumping principally to deep wells.

While some plants have been installed where the air-lift principle was used to pump water through horizontal pipes, the air in passing through such pipe is likely to collect along the top of the pipe, allowing a large space in the lower portion of the pipe for the water to slip past the air down the pipe, and thus greatly reduce the efficiency of operation. Also in a horizontal pipe it is not practicable to utilize the expansion of the

air bubble as it approaches the surface where the pressure upon it decreases. If it is desired to convey the water to a point distant horizontally from the well, the eduction pipe may be carried vertically to a height sufficient to allow the water to flow to such distant point by gravity.

While aeration of the water is frequently an advantage, under some circumstances it is a disadvantage, and in such cases the air lift is not advisable. For instance, it probably promotes the rusting of the eduction pipe and in some cases causes a deposit of salts which clog the water passages, especially in the foot piece. It is also believed by some that the air causes an excess growth of algæ.

ART. 63. SELECTION OF PUMP. STATION DETAILS

The type of pumping plant selected for a given service should be that which will be most suitable from the point of view, first, of its ability to pump water under the conditions existing and anticipated, and without any "engine troubles"; and, second, of economy of combined first cost and operation. The procedure is generally to determine what types and variations thereof will perform the service effectively, and then compare these on the basis of cost.

The principal factors to be considered are (1) the amount of water to be handled per day, the head against which it is to be pumped, and whether or not these are constant; (2) what quantity and head must be provided for in the future; (3) the cost of fuel of various kinds—coal, gas, oil, etc., and of electricity, and the probabilities as to changes of price within a few years; (4) the space available and location of pump—whether in a well-casing, in a pump well below the ground surface, on land subject to occasional flooding, in the heart of a city, whether or not accessible for delivery of coal or obtaining electric current, etc.; (5) whether solid foundations can be provided at low cost; (6) importance of absolute reliability of service; (7) whether the new unit is the first to be installed or is to be added to a plant already in service, and whether it will be used for every-day service, or as a stand-by for fire or other emergency use.

Effectiveness for Service. For a pump handling more than

about two million gallons per day in continuous service, a steam plant is generally considered the most reliable. If the amount is less than one million gallons, steam is still more reliable than internal-combustion engines with equally intelligent supervision, but as less expensive engineers are generally employed, this superiority over internal-combustion and electric motors diminishes. The former are somewhat liable to unexpected stoppage for various reasons with which automobile owners are familiar; while electric power may fail occasionally because of broken wires or power-plant troubles over which the pump user has no control. Any of these powers can be used for any head, but as this increases the advantage of steam becomes greater.

If the head or consumption varies from minute to minute, as in direct pumping, steam is preferable, since both internal-combustion and electric motors work most satisfactorily at a certain fixed rate of motion. On the other hand, if the pump is to be put into service on short notice, as for fire pressure, full power can be applied more promptly by one of the other types of motors, since it is generally impracticable to keep continuously at full pressure sufficient extra steam for this purpose.

Of the several varieties of steam-pumping engines there is little choice as to reliability, choice being based largely upon economy; but the high-duty engines are more complicated than the low, and so more expert care is required to keep them in condition. For example, a low-duty Worthington pumping engine contains 28 major moving parts; a high-duty Worthington 90 parts; and a horizontal crank-and-flywheel 100 parts.

For fire service, the National Board of Fire Underwriters considers the following as reliable, precedence being in the order named: Centrifugal or reciprocating pumps driven by steam; the same driven by electric motor; pumps operated by water power; centrifugal or reciprocating pumps operated by internal-combustion engines, duplicate ignition parts to be always on hand for each engine, and adequate provision made for starting engines cold at least six times in rapid succession.

If the pump is to lift water from a well-casing, the choice is limited to a form of bucket pump, a small turbine pump, and

the air lift. Any of these are fairly reliable, but repairs require a removal of the appliance from the well, which requires considerable time, and the air lift has no moving parts below ground to get out of order. If the wells are close together, steam may be used, distributed from a central station, or air from a central compressor plant; but if they are widely separated, or the power required is small, individual electric motors or internal-combustion engines are preferable for pumps, although compressed air can be distributed for considerable distances.

The importance of reliability of a given pump varies with the relative number and capacity of other pumps in the same service, and the amount of water storage between pump and consumer. If there is no such storage, reliability of service is supremely important. If storage sufficient for several days can be counted upon, a few hours' stoppage of pumping is not so important.

If pumping is from a river subject to a rise of 20 feet or more, the pump must generally be in a watertight well at least as deep as the height of maximum flood above low water less 15 feet. Under such conditions the horizontal space occupied should be a minimum, while height is an advantage as bringing the motor end of the mechanism above possible floods or leakage into the well. Hence a vertical pumping engine, or a centrifugal pump operated from a motor elevated above reach of flood, is to be preferred. Also if the plant is located in the heart of a city or any other locality where land is expensive or the amount obtainable is limited, the floor space occupied by the plant is an important consideration. A vertical pumping engine generally requires a more solid foundation than a horizontal one. In the case of restricted area there may not be room for a boiler plant, or there may be objection to the smoke, ashes, coal hauling, etc., in a select neighborhood, when electric power, or a Diesel or other high-duty internal-combustion engine may be advisable. The same may also be the solution if the plant is located at a point not easily accessible for a coal siding or truck hauling.

Any pump may at some time meet with an accident, and all are subject to wear in parts that should be adjusted or replaced

from time to time. Also in most cities the consumption varies from week to week and increases from year to year. For these reasons it is advisable to have at least two pumps in the plant, and to increase the number to three or four as the plant grows. If the present consumption seldom exceeds one million gallons a day, a two-million gallon pump (to operate part time only), and a three-million might be installed, the latter used as a reserve until the consumption had increased about 50 per cent, when it would be put into regular service and the other held in reserve, being still able to pump that amount by sixteen hours operation per day. Or two two-million gallon pumps might be installed, a high duty for regular service and a low duty and cheaper one for reserve; the latter to be replaced by a three- or four-million pump when the consumption required. It should be remembered that while continuous twenty-four hour pumping is more economical of fuel than banking fires every night, one shift of engineer and fireman is much cheaper than the two or three shifts that would be required for more than nine or eighteen hours' pumping. But of course in direct-pressure systems the pumps must be kept operating continuously, and in addition to one reserve pump there should be sufficient pump capacity to supply the total maximum of combined normal and fire consumption. It is generally economical to have one pump with a capacity of the maximum normal consumption, a reserve with the same capacity, and another reserve capable of increasing pressure and quantity to meet fire needs; the last operated electrically or by a reliable internal combustion engine, so that it can be placed in full operation on five minutes' notice.

As to boilers, the fire underwriters give as a minimum reserve capacity one-fourth the entire capacity required to operate all the pumps that will be needed at time of maximum consumption, there being at least one boiler so held in reserve. In a small plant it may be most economical to provide two boilers of equal capacity, to be used alternately, utilizing the alternation of use for cleaning of flues, etc. For larger plants the number may be increased up to five (and even more for the largest), one held in reserve, and one or two used for peak loads when the consumption varies, as in a direct or direct-indirect system.

For boiler pressure of 150 pounds or under and in small plants, the return-tubular boiler is probably to be preferred; but for higher pressures and large plants the water-tube boiler is almost necessary.

Economy. More difficult than the selection for effectiveness and reliability is that on the basis of economy. Ultimate economy involves the first cost of the entire plant, including buildings; the cost of adequate excess capacity to provide for emergencies and increase in consumption; the number of years before it will wear out, become too small, or become economically out of date (depreciation and obsolescence); the cost of keeping it in repair and prime working condition (maintenance); the cost of fuel and of attendance (operation); the value of the plant as a reserve when dropped from regular service; the cost of land, foundations, and buildings required to support and contain it. To combine all of these, each must be reduced to the basis either of annual cost or of capital investment; that is, a percentage of the lump sum cost is taken equal to the sum of the interest on such cost and the annual depreciation and obsolescence, which sum is considered as the annual cost of this item, and this and the actual annual maintenance and operation costs are added together to obtain the total annual costs. The plant giving the lowest total annual cost is the most economical. Or the sum may be calculated which, at a certain interest rate, will yield the annual operating and maintenance costs and provide for depreciation, and this sum be added to all the lump costs to give a total capitalized cost for comparison. The former is the more common method.

In general, the plant of a given type that develops the highest duty costs the most and also requires more expensive operators and more minor repairs and constant attention. It is apparent that if such a plant is purchased, the additional cost is wasted if the high efficiency is not maintained continuously in service. Duty of a plant ultimately reduces to the basis of fuel used, and extra cost of high duty must be more than offset by cost of fuel saved to be economical. Therefore the cheaper the fuel the less can be paid for high duty. High-duty engines are seldom economical in a district where coal is very cheap.

The same applies to comparing types. If a gasoline pumping plant costs much less than a steam plant (including boilers, boiler house, etc.), it may or may not be more economical, depending upon the relative cost per year of the coal and gasoline, respectively, and of the operating force and repairs. As the cost of gasoline rises, the gasoline engine loses more of its advantage of low first cost.

In making these analyses, care must be taken to include all elements, such as cost of land and housing, of foundations, value of plant when dropped from regular active service; also to consider the possibility of future increase or decrease in the cost of fuel, and how the anticipated increase in consumption can be provided for. As to the last, all pumping plants operate more efficiently at full capacity that at part; a steam plant may use 80 per cent as much steam when performing only one-third as much work. Consequently it is better to pump few hours at full capacity than two or three times as long at half or third capacity. Coal is in most cases but 30 to 45 per cent of the total operating cost, the percentage decreasing as the size of the plant decreases. For small plants (1,000,000 gallons or less) the fuel is a small item and other costs will largely control.

The annual cost of fuel used in a steam plant equals $\frac{990TCP}{DE}$, in which T is the number of hours during the year that pumping is required (plus an allowance for banking fires); C is the cost of coal per ton of 2000 pounds; P is the average indicated horse-power of the engine; D is the average service duty per 1000 pounds of steam, and E is the evaporation factor of the boiler (approximately 8 pounds of water per pound of good bituminous coal); or DE is the station duty in terms of pounds of coal.

The cost of boilers decreases with the duty of the pump, thus tending to offset the higher cost of high-duty pumps. Thus, allowing 10 square feet of boiler heating surface per boiler horse-power, 1.63 boiler horse-power per pump horse-power is required if the duty is 40 million per 1000 pounds of steam, but only 0.33 boiler horse-power for a duty of 200 million.

An illustration of a comparison of three types of steam

pumping engines for a certain service is given herewith. The figures and conclusions apply, of course, only to this particular case:

COMPARISON OF CONSTRUCTION AND ANNUAL COST OF THREE TYPES OF PUMPS

From paper by Nicholas S. Hill before Municipal Engineers of the City of New York.

Items.	Type of Pumping Engines.		
	Horizontal Cross Compound.	Horizontal Triple Expansion.	Vertical Triple Expansion.
Cost of construction:			
Engines (four)	$237,000	$304,000	$425,000
Engine foundations	8,500	12,600	
Boilers and foundations required for complete station	45,000	41,500	41,500
Superheater	11,000	11,000	11,000
Economizer	16,000	16,000	16,000
Chimney (radial brick)	10,000	10,000	10,000
Buildings required for complete station	60,000	72,000	72,600
Centrifugal pumps	18,000	18,000	18,000
Four valve engines for centrifugal pumps	24,500	24,500	24,500
Foundations for centrifugal pump and engines	1,000	1,000	1,000
Generators, engine switch-board and foundations	20,300	20,300	20,800
Steam piping	10,000	10,000	10,000
Total cost of station; above basement floor; not including sub-foundation, suction intake, discharge piping	461,300	540,900	650,400
Yearly cost of pumping:			
Fuel	90,000	85,000	80,000
Wages	20,000	20,000	20,000
Repairs to machinery	5,400	6,300	8,100
Supplies, oil and waste	5,000	5.500	5,500
Interest and depreciation	39,200	46.000	55,300
Refunding sinking fund	13,950	16,350	19,670
Total yearly cost of pumping	173,550	179,150	188,570

If, as is usually the case, the pump is not to be operated all the time, the comparison of fuel, wages, and supplies should be based on the actual pumping done. Thus, in the above calcula-

tion, if the assumption was on a basis of 24-hour pumping, and another should be made based on twelve hours of pumping per day, the yearly costs would be as follows:

Fuel (including 5 per cent for banking fires)	$47,250	$44,625	$42,000
Wages	10,000	10,000	10,000
Repairs to machinery	2,700	3,150	4,050
Supplies, oils and waste	2,500	2.750	2,750
Interest and depreciation	39,200	46,000	55,300
Refunding sinking fund	13,950	16,350	19,670
Total yearly cost of pumping	115,600	122,875	133,770

This is still more favorable to the cheapest machine. Incidentally it shows that the cost per million gallons pumped is increased one-third by using the plant only half time but at full capacity while in service.

In selecting a reserve unit, the same principle is employed, but the pump is assumed to be operating only a part of the time —say 10 per cent. It follows that first cost is the chief item in economy for such a pump; but even more important is it that the pump be reliable and one that will suffer least in efficiency and dependableness by non-use.

In comparing the coal consumption of different types of pumps, it may be tentatively assumed that the duty per 1000 pounds of steam will run about as follows:

Vertical, triple expansion, crank and flywheel	150 to 200 million
Horizontal, cross-compound, crank and flywheel	110 to 150 million
Horizontal, duplex, direct-acting, triple	75 to 110 million
Horizontal, duplex, direct-acting, compound	50 to 70 million
Centrifugal pumps, steam turbine driven	70 to 90 million
Centrifugal pumps, engine driven	130 to 160 million

The nearest approach to a perfect pumping plant now available for conditions found in the average plant of large capacity would probably consist of vertical, triple expansion pumping engines, with long stroke and maximum piston travel of 200 feet per minute, steam jacketed, using superheated steam at 175 pounds gage pressure; water-tube boilers with mechanical stokers, a draught of at least 0.8 inch of water, with feed water heated by economizers and heaters.

But for smaller plants the refinements just named would cost more than they could possibly return in fuel saved. J. N. Chester considers that for a principal pumping unit, under average conditions, the compound condensing is the best for daily capacities of between one and two million gallons; direct-acting triple expansion for 2,000,000 to 4,500,000 gallons; cross compound for 4,500,000 gallons to 8,000,000 gallons; and the vertical triple expansion above this. Also that the station duty per 100 pounds of coal that these should give in service varies between 20 and 30 million for the first; between 40 and 50 for the second; 50 and 70 for the third, and 70 and 100 for the fourth. (These are about half the test duties obtained.) For reserve units he recommends compound condensing up to 3 million capacity; direct-acting triple between 3,000,000 and 5,500,000, and steam-turbine driven centrifugal for all larger sizes.

For less than one or two million gallons per day capacity the internal combustion engine should be considered, driving a power pump. Distillate, kerosene, and gasoline differ little in their heat values, but the first two cannot be used by all engines. Their relative costs vary considerably from time to time. A semi-Diesel engine will use approximately $\frac{3}{4}$ to 1 pint of fuel oil per brake horse-power. The semi-Diesel engine is generally used for small plants, and the Diesel for those of more than 100 to 150 h.p. The latter has a better fuel economy. The cost of an internal-combustion engine and pump is generally more than that of a steam pumping engine alone, but not if the boiler and steam piping be considered. The oil engine cannot carry so much overload as the steam, its depreciation and repair rates are higher, but the attendance labor is less, especially in small plants. As stated before, the present makes of oil engine are less reliable than a steam engine, and more skill is required to keep them running smoothly.

STATION DETAILS

In planning the pumping station it should of course be made large enough and high enough to contain the pumps, boilers, and all auxiliary apparatus. When possible it is well to delay the

designing of the building until the contract for the pumping plant has been awarded, and make it to fit this. Give due consideration to the need of duplicating the plant in the future, by either making the station at once of ample size for this or providing for adding wings or extensions later. The boiler room should be separated as completely as possible from the engine room to prevent ashes from the former entering the air of the latter.

The coal house should be as near the fire doors as possible, to reduce the handling of coal. A good plan is to place a long coal room just in front of the boilers, with only sufficient room between to permit of using the rake, flue cleaners, and other boiler tools. The boiler room should be well built-in to prevent unnecessary radiation of heat in winter. Boilers have been known to have their efficiency decreased 5 per cent or more by cold weather. Facilities for delivering coal to the station should not be neglected. It should be near a railroad, canal, or navigable river if possible. If the coal come by rail it may be economical to furnish a siding into the coal room or along its outer side; and if by water, to provide a dock and hoisting-derrick for filling the bins.

Foundations should be given most careful attention. Where it is possible to reach rock, a solid concrete foundation for the pumps should be carried down to this, or at least walls or piers connected at the top by heavy reinforced slabs. If rock is not within reach, piles driven to hardpan may be used, their heads extending into a heavy slab of concrete forming the foundation. Piles not reaching hardpan or rock are liable to settle owing to the vibration of the engine. If the foundation of an engine rests on earth, it should be spread so that the load on the earth shall not exceed about 800 pounds per square foot, to allow for this vibration.

All piping should be planned before the foundation is made, so that provision may be made in the latter for any pipes below the floor level. The engine makers generally furnish a wooden template for setting the anchor bolts in the foundation. If the engine and pump are on different foundations, the greatest care

should be taken to see that the two are properly adjusted to bring the couplings, connecting gear, or belt pulleys into exact alignment.

The boilers being motionless and largely self-contained, their foundations can be calculated on a basis of 2500 to 3000 pounds per square foot of earth (the weight of water in the boilers not being forgotten), although here also, of course, rock is preferable. The chimney foundation has already been referred to.

The steam piping is generally designed by the contractors furnishing the pumping plant or boiler plant. It should be as direct as possible, with no unnecessary angles. If the pipes are too small, energy is lost in friction of the steam passing through them; if they are too large, there is more radiating surface and more time consumed by the steam in getting from boiler to pump, and therefore more loss of heat. The pipes should be completely covered, flanges and all, with closely fitting heat insulating material. The loss of heat from bare steam pipes is about 3 B.t.u. per square foot of pipe surface per hour for each degree difference in temperature, but this may be reduced to 0.3 to 0.7 B.t.u. by good insulation. Provision for longitudinal movement due to expansion and contraction should be made in the supports of the steam pipe. The movement may be taken at about 1 inch for each 50 feet of pipe.

In the pump room should be placed gages showing steam pressure in the boilers, water pressure in the discharge pipe and vacuum in the suction, and the amount of discharge as measured by a meter (generally of the venturi type). There should be recording as well as reading gages. There should be pump counters to record the number of strokes of each engine.

The boiler room should be well ventilated, and in large plants is generally provided with facilities for handling both coal and ashes by mechanical stokers and ash conveyors.

The engine room should be light and well ventilated. Walls of white or light enameled brick or other non-absorptive substance from which oil can be removed easily are advisable.

CHAPTER XIII

DAMS AND EMBANKMENTS

ART. 64. MATERIALS FOR CONSTRUCTION

DAMS have been built of earth, loose rock ("rock-fill"), timber, iron, and steel, and of concrete, stone, and brick masonry; and cores of timber, iron, and masonry have been used in many dams of the first and second classes. Of these, masonry is the most substantial and tightest, timber probably comes next (within the lifetime of the timber), and loose rock and earth last. Iron and steel have been used for few other than movable dams, except as a facing or core. A poorly designed or constructed masonry dam may of course be weaker or more porous than a timber or earthen one; but the above order will hold generally for properly designed and well-built dams.

Timber dams are most applicable for use as weir dams or overflow dams—that is, those over whose crests water flows—because of the tendency of the timber to rot if alternately wet and dry. Earth dams, on the other hand, will surely fail if any water whatever flows over them. Rock-fill dams are not intended to serve as weirs, and, although an occasional small flow over their crests may not prove fatal, should not be designed to serve as such.

Masonry is the only material adaptable to weir dams of permanent construction. It is also used for high-crest dams which are not intended to pass water, when absolute stability or tightness is desired, or when the height of the dam exceeds 60 to 75 feet (although earth dams have been constructed much higher; as the San Leandro, Cal., dam, 125 feet high). Earth dams are especially adapted to locations where there is not a sufficiently firm foundation to support a masonry dam, or where the cost of

one would be excessive; as in the case of the Honey Lake valley Cal., dam, 118 feet high, which was so inaccessible that haulage brought the cost of cement up to $8.25 per barrel. If the distance to bed rock or excessive cost render the use of masonry inadvisable, and an overflow dam is required, a timber dam is probably the only alternative; but if the work is extensive, the dam more than 40 or 50 feet high, or a permanent construction desired, masonry must be used at whatever cost.

It may sometimes be desirable, for financial reasons or to give time for a more thorough study of the run-off or other conditions prior to making final plans, to construct a temporary dam of timber a little above the best location for a masonry dam, and build a masonry dam at this location sometime within ten or twenty years, for which length of time a well-constructed timber dam should be perfectly sound.

Art. 65. Foundations

The most important feature of a masonry dam is the foundation. The great majority of the serious failures of such dams have been caused by a weak or otherwise inferior foundation and not by weakness of the structure itself, as such.* In the case of dams of other materials also, the foundation is very important. A location for a dam should therefore never be definitely selected until the foundation has been thoroughly examined.

A foundation for any dam should be practically tight. Probably no rock is perfectly impervious to water under a high head, but it should be definitely ascertained that there are no seams by which water could escape under the dam or around its ends, and that not sufficient water can seep through the rock to a clay seam below to cause an eroding velocity therein. If there are such seams, a trench should be carried downward, or extended at the end of the dam, until solid rock without seams is reached. If there is a clay seam under the top stratum of rock, unless the latter is very thick or the dam is low, the masonry

* Edward Wegmann knows of but five masonry dams more than 49 feet high that have failed, and bad foundation was the cause in each of these, assisted by poor masonry in one case.

dam (or core wall, if the dam itself is not of masonry) should be carried to rock below such clay seam. Some of the most disastrous failures of masonry dams have been caused by the washing out of such clay seams and consequent caving down of the superimposed rock, this being caused by the pressure of the water behind the dam or the back-wash under the toe of the dam occasioned by the storm overflow.

Before beginning a masonry dam, all soil above bed rock must be removed, all boulders and loose or seamy rock, thin shells, and everything down to solid bed rock. The higher the dam, the more important this work, of course. Since masonry is practically inflexible,* an unyielding foundation is necessary if there are to be no unknown or unprovided-for stresses in the structure that may cause its rupture.

If, when the solid rock is reached, there are found to be a few joint or other seams but the rock is otherwise sound, these may sometimes be filled with cement mortar worked in from the surface or grout forced in under pressure through drilled holes; but this should be relied on only after a most thorough investigation and by advice of experts.

Only by the excavation of the overlying soil and loose rock can the character of the foundation be known with certainty; but it is generally desirable to have an approximate knowledge of it beforehand as a guide in preparing plans and letting contracts. This can be obtained by sinking test pits through soil or by wash borings (the former is much preferable), and by core borings in rock. The last should be filled with cement where they extend below the final excavation, to prevent leakage through them.

Comparatively small and low masonry dams may be set upon heavy timber foundations resting upon piles or a solid hardpan foundation; but only the greatest care to prevent undermining of the foundation and a large factor of safety in the masonry can render such a dam secure. Close sheet-piling should be carried entirely across the upper end of the foundation, placed

* There is a very slight flexibility to masonry, but safety demands that it be not called into play intentionally.

in a trench which has been dug down to and a foot or two into hardpan or clay, and puddle well rammed into the trench on each side of the sheeting. The sheeting should be carried up 2 or 3 feet higher than the foundation and the space between it and the masonry filled with concrete or puddle. The foundation should be carried down-stream from the dam for a distance at least equal to the height of the dam, to act as an apron to prevent the falling water from undermining the foundation. At the end of the apron should be placed sheet-piling to prevent undermining of the apron, but openings of small area should be left in this to permit the escape of any water forced under the apron from above the dam, which might otherwise force up the

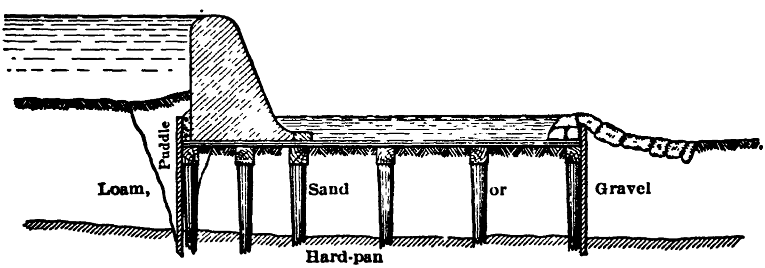


Fig. 67.—Masonry Dam on Timber Foundation.

apron by hydrostatic pressure. In the case of a weir dam or spillway, a dam 2 or 3 feet high is sometimes placed at the end of the apron to form a water cushion at the face of the dam, as shown in the illustration.

It should be borne in mind that such a foundation has to resist not only the weight of the dam, but also the pressure at the toe due to the pressure of water behind the dam, and the horizontal pressure of this water also. Also the apron must resist the impact of extreme floods overflowing the dam. Most important is it that the cut-off wall back of the dam be tight; for any leakage under the dam, under the pressure of the water above, would result in erosion which would sooner or later undermine the foundation and probably reduce the stability of the foundation piling.

When a masonry dam rests upon rock, the top of the rock

should be left very rough and uneven. If it is smooth from being water-worn or glacial action, or is a smooth natural bed, a cut-off trench should be excavated at the up-stream edge (heel) of the dam to prevent water penetrating under the foundation. Also a trench should be excavated at the toe of the dam to prevent its sliding on the foundation, unless the entire foundation is so excavated in rock as to effect this purpose.

A rock-fill dam should rest upon rock as safe as that for a masonry dam; for more or less water finds its way through such dams, which would erode any earth under the dam and cause it to settle.

Timber dams are preferably placed on rock; but are often placed on hardpan or clay, in which case one or several cut-offs of sheet-piling and puddle should be provided, each carried continuously for the entire length of the dam and put down to a depth which will prevent water being forced under it by the greatest head of water that can form behind the dam.

While masonry dams should always rest on rock if possible, earth dams, or embankments, should never do so if it can be avoided; for it is practically impossible to prevent water following the surface of rock with which earth is in contact. Where earth must rest upon rock, the rock surface should be made rough—preferably cut into trenches or "saw teeth" parallel to the axis of the dam and the embankment packed solidly into these; and in addition a water-tight core wall should be carried the entire length of the dam and up to the highest level reached by the impounded water.

Even rock, however, is a better foundation for an earth dam than porous soil, and all such as well as all vegetable matter (which will decay and leave voids) should be removed from under an earth dam. "Porous" is a relative term, as water will probably seep slowly through any soil; but the movement should be so slow that no erosion or movement of earth particles will take place. If such erosion once begins, the dam is probably doomed unless it can be stopped. The rate of motion depends upon the head of water on the soil, the nature of it, and the distance through which the water must move before finding a

free outlet. The last can be increased by increasing the width of the base of the dam, and this is frequently done, embankment slopes as flat as 33 to 1 having been employed in some cases, although between 2 and 5 to 1 are more common. (With both embankment slopes 5 to 1 and a 30-foot top width, the distance to be traveled by water which is passing under the embankment will be 30 feet plus 10 times the height of the embankment, or 530 feet for a dam 50 feet high.)

If the formation below the top soil is porous but solid (like clean gravel) and rests upon hardpan or clay, seepage under the dam may be prevented by cut-off walls of sheet piling and clay puddle, or of concrete, carried down into the impervious material; but even with these, chief reliance should be placed in a tight core well.

In any case, the surface of the earth forming the foundation should be furrowed or trenched longitudinally of the dam, to secure a better bonding between foundation and embankment.

Art. 66. Designing Masonry Dams

A masonry dam, of either stone masonry or concrete, is assumed to be a rigid, monolithic mass. It may yield in one of three ways (in addition to yielding of the foundation): It may be overturned as a whole, revolving about the toe, or the part above any horizontal plane may so revolve; it may slide as a whole on the foundation, or any part may slide on the part below it; or it may yield by crushing of the masonry. The forces acting are: The weight of the masonry; the static pressure of the water; the dynamic effect of a river current, waves, floating logs, ice, or other floating solid matters; the pressure of ice in expanding; suction on the face due to water overflowing the crest; and the pressure of wind.

The static pressure of water on any vertical element of the dam 1 foot long, in pounds is $31.25\ h(h+d)$, in which h is the height of dam or of the part below the water surface, and d the depth of flow over its crest, both in feet. This is exerted normal to the surface in contact with water. If there is any communication between the water behind the dam and the under side of

the dam (which the heel cut-off is designed to prevent, but may not), there is an upward pressure on the base of the dam equal to 62.5 $(h+d)$ pounds per square foot; but the area over which such pressure is exerted will generally be confined to small channels in the mortar, or the areas over seams through which water finds access to the foundation. Both of these should be negligible in area but some designers assume such pressure on 15 to 25 per cent of the area of the base.

The pressure due to a river current will seldom be appreciable in amount, except at the very crest, and here will probably never exceed 100 pounds per square foot. It can be neglected in practical designs, except perhaps for flash boards. The impact due to waves will probably never exceed 3000 pounds per square foot in the largest reservoirs, and will seldom exceed 1000 pounds. This is exerted against the crest of the dam only, as is also the impact of floating logs and ice, which equals $\frac{Wv}{gt}$. It is not probable that any such matter will strike the dam when there is a depth over the crest of more than 3 feet; at which depth the velocity of flow over the crest will be about 8 feet per second, which would make the energy of the moving logs or ice about W, and the impact about $\frac{W}{4}$. A tree or log would not, therefore, probably strike a dam-crest with an impact greater than 4500 pounds; and probably a height of 2 feet on the crest and 2250 pounds impact would be high enough for most cases.

Ice may exert a pressure of 43,000 pounds per lineal foot (report on Quaker Bridge dam); but there appears to be only one case on record where a dam has failed from this cause—at Minneapolis, where a retaining wall paralleled the dam 350 feet away, the dam was moved a foot at water level by ice 4 feet thick. But in an ordinary reservoir the ice will move away from the dam or shear before exerting a pressure nearly as great as this.

The impacts and ice pressure referred to should be provided for in designing the thickness of the crest of the dam, but may be disregarded in designing the dam as a whole.

The " suction " exerted by water flowing over a dam cannot exceed 14.7 pounds per square inch, nor can the total amount exceed the total horizontal impulse of the water flowing over the dam at the time being. It can be avoided altogether by any construction that gives free access for air to the under side of the sheet of overflowing water, which construction should be provided.

In addition to the external pressures, changes in volume due to temperature cause stresses in a dam that tend to cause vertical cracks. In a masonry dam such cracks generally form on the surface; but observations made on the Boonton, N. J., dam indicate that there is very little change of temperature in the inner part of heavy masonry. Cracks so formed are approximately vertical and have little direct effect upon the stability of the dam, but may have upon the tightness. Several recent large masonry dams have been provided with expansion joints at intervals of about 80 feet which permit longitudinal motion of the dam in sections without causing leakage.

The forces acting to overturn or rupture a dam may be resisted by weight, friction, or strength to resist tensile, compression and shearing stresses, or by combinations of these. Masonry dams must be considered to have no tensile strength except such as may be contributed by steel reinforcement in reinforced concrete. In steel dams the weight is of little importance.

The compressive strength of masonry is found to depend to only a minor degree upon that of the stone, but to be largely affected by the character of mortar and of bond. The limits of safe pressure are given as follows by Baker:

	Tons per sq. ft.
For concrete	5 to 15
For rubble	10 to 15
For squared stone	15 to 20
For limestone ashlar	20 to 25
For granite ashlar	30

The maximum pressure on the granite masonry of the towers of the Brooklyn bridge is about 28½ tons per square

foot (about 400 pounds pressure per square inch), and on the limestone masonry about 10 tons per square foot (125 pounds per square inch). The limestone masonry in the towers of the Niagara suspension bridge failed under 36 tons per square foot and were taken down, but this masonry was not well executed.

The weight of masonry for any given case should be ascertained by actual test. Concrete will generally weigh between 140 and 160 pounds per cubic foot; granite or limestone rubble 150 to 155 pounds. The coefficient of friction may generally be taken at .60 to .65, or the friction at 85 to 100 pounds per cubic foot of masonry.

The strength of masonry to resist shearing stresses is not known, but is apparently more than ample in any dam which is sufficiently strong otherwise and has sufficient weight.

Masonry dams may rely upon their weight to prevent overturning, upon arch action, or, when of reinforced concrete, upon beam action. The first are called gravity dams, and are by far the most numerous. The second are called arched dams, and such a dam may consist of one large arch or be multiple-arched. The third class are hollow or cellular dams.

GRAVITY DAMS

In a masonry dam the maximum pressure must not at any point exceed the safe crushing strength of the masonry. It can be demonstrated that safety against crushing at the toe and against overturning demands that the resultant of all forces acting at any time, whether the dam be empty or full, must fall within the middle third of the base. In each calculation the pressure caused by the greatest possible elevation of water surface must be considered. Consideration of these points must be applied not only to the dam as a whole, but to every possible section of dam from the crest down, each considered as a dam resting upon the masonry below it. Finally, economy calls for the use of the minimum amount of masonry, with due consideration of the cost of building masonry to forms other than rectilinear

In calculating sections of gravity dams, it is most convenient to use the units lineal foot (measured along the axis of the dam), square foot, and cubic foot, and to consider a section of dam 1 foot long.

Of all the forces acting on a dam, the static pressure of the water behind it is much the most important, and the design will first be made considering this alone, and later modified if necessary to provide for the other forces.

Total resistance to shear and compression vary with the width of the dam, and friction and resistance to overturning vary with the weight, which in turn varies with width and height.

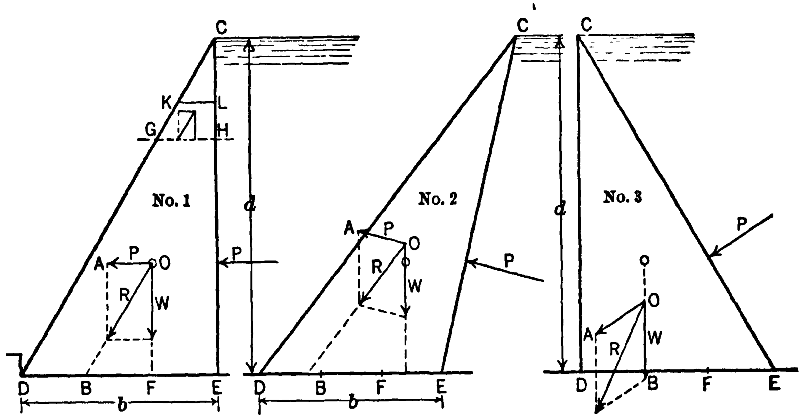


FIG. 68.—Calculation of Triangular Dam Section.

Consequently the width should vary as the pressure. Static pressure varies directly as depth below water surface, being zero at such surface. Therefore the theoretical section of dam calculated to just resist the water pressure would be more or less approximately a triangle with its vertex at the surface of the highest possible water level. There are three possible conditions—the back or up-stream surface vertical, inclined up-stream from the vertical, and down-stream from the vertical.

Consider first the vertical-back dam, No. 1, a section of a masonry dam 1 foot long, the height being d feet and the width b feet. Let w be the weight of a cubic foot of water $= 62\frac{1}{2}$ pounds; and m the weight of a cubic foot of masonry. Then the pressure

of water P equals $\frac{1}{2}d^2w$, applied at a point $\frac{d}{3}$ above the base, and horizontal in direction. The weight of the masonry, W, equals $\frac{dbm}{2}$, acting vertically through the center of gravity O, the line of action of the force intersecting the base at F and FE being $\frac{b}{3}$. Therefore when the reservoir is empty ($P=0$), W is the resultant and it just falls at the edge of the middle third of the base. When the reservoir is full, the resultant R must not fall beyond BF, the middle third. If it falls at B, $P:W=b:d$. Or, substituting values of P and W, $\frac{d^2w}{2}:\frac{dbm}{2}=b:d$, or $b=d\sqrt{\frac{w}{m}}$. That is, as a general law, the width of base of a triangular section must be at least the maximum depth of water divided by the square root of the specific gravity of the masonry.

When resultant R falls at B, the pressure at E is zero, and that at D is twice the average for the entire base, or $\frac{2R}{b}$. This must not exceed the safe crushing strength of the masonry; if it does, the base b must be increased or a stronger class of masonry used.

The coefficient of friction of dam foundation, multiplied by W, must exceed P; or else some additional resistance to sliding must be provided, as a shelf in the rock at D.

Making the same analysis concerning any section CGH considered as a dam resting on a foundation GH, we find the same conditions to hold. Here, however, the tendency to sliding is resisted by the combined strength of the masonry to resist shearing and the friction on the plane GH. It is believed that no masonry dam has ever failed by shearing, and this need not therefore be considered. Dams have, however, been pushed down-stream by sliding on their foundation.

C is assumed to be the greatest height the water will ever reach. If there is a possibility that the dam will ever be overtopped by a flood, let the height of the maximum overflow be CL; then the section of the dam should be $EDKL$. (This is

not theoretically correct, since *P* will be less, its point of application lower, and *W* will be less. But it errs on the side of safety.

Considering section No. 2, we see that when the dam is empty and *W* is the total force acting, this falls outside the middle third of the base, and according to the assumption this section therefore cannot be used.

Consider section No. 3, in which the up-stream face inclines so far down-stream that the other face *CD* is vertical. Then when the dam is empty the resultant passes through *B*. If the line of pressure *P* cuts the base at the left of *B*, the resultant *R* will fall outside the middle third. This line of pressure must pass through *B* or to the right of it. To pass through *B*, therefore, *b* must equal *d*. That is, if the face down-stream is vertical the base must be at least equal to the altitude. But with the vertical face up-stream, the base need be only equal to the altitude divided by the square root of the specific gravity of the masonry—that is, need be only about two-thirds as wide. The section No. 3 is therefore extravagant of masonry; and sections intermediate between No. 1 and No. 3 are also intermediate in the amount of unnecessary area.

The top of a dam must be able to resist the impact of current, floating wood or ice, waves, etc., and for this reason the top of the section must have sufficient width to furnish the necessary resistance. Probably 12 inches if of concrete, or 24 inches if of stone masonry, is the least width to be given to even the lowest dam. There is no general rule for width of top, but it should be adapted to resist such impacts as have been named. These may be 3000 pounds per square foot, and this must be resisted by the tensile and shearing strength of the joints of masonry or of the concrete, plus the friction due to the masonry above water level. If the top of the dam is 10 feet wide, the masonry weighs 160 pounds per cubic foot, and the friction coefficient is .65, a height of 2.9 feet above a joint or plane where the blow is delivered would of itself prevent sliding along such joint. For a small reservoir 12 to 18 inches would be sufficient; or 2 or 3 feet if the crest be made 5 feet wide. It is desirable to carry the masonry to this height above the highest water.

The tensile and shearing strength is not definitely known, and such as it is will act as a factor of safety. An overflow dam

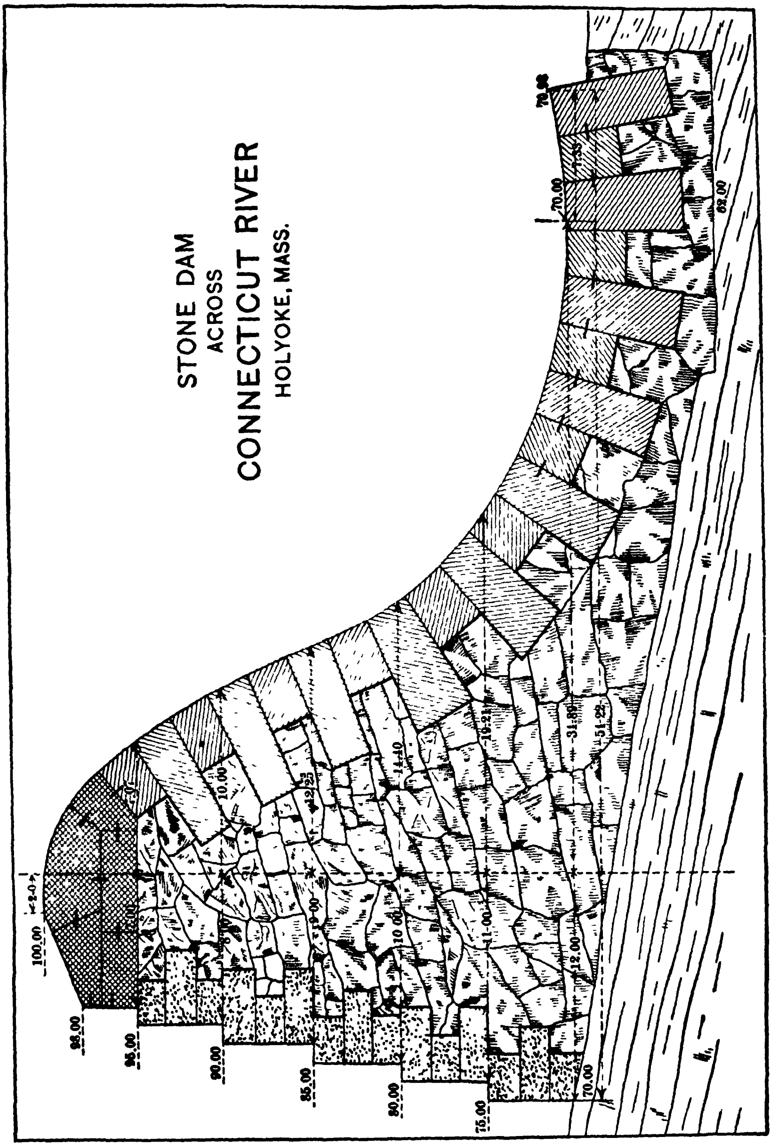


FIG. 69.—Weir Dam at Holyoke, Mass.

or spillway must resist this dynamic pressure plus the static pressure due to the depth of overflow. The crest is generally

made broad with heavy crest stones, when the dam is of stone masonry. In addition, it is well to clamp the crest-stones, or cap stones, to each other and to the dam beneath, and also to slope the crest toward the back of the dam, to cause floating objects to strike a glancing rather than a straight blow.

For high-crest dams there is no established rule or law for determining the thickness of the crest. It generally varies from one-third the height for low dams to one-tenth the height for dams more than 100 feet high. This thickness should extend undiminished down to where it joins the triangular section, or even batter somewhat, as in Fig. 70, *A*. If the break in line of

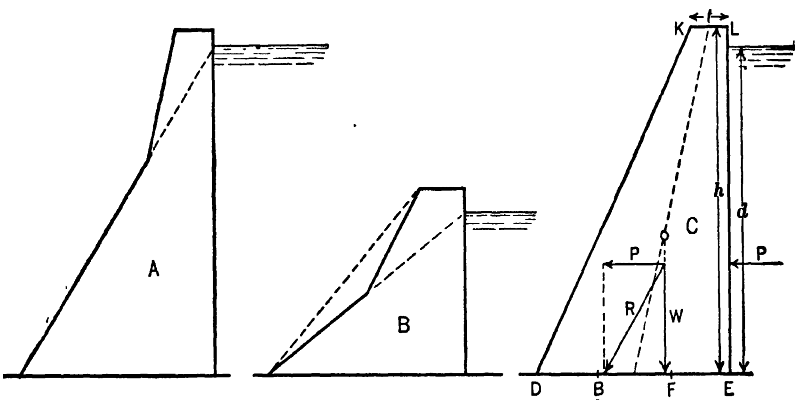


FIG. 70.—Practical Sections of Gravity Dams.

the lower face of the dam so caused comes less than $\frac{2}{3}d$ above the base, it is often well to make this face a straight line, as shown by the broken line in *B*. This gives a better appearance, and is somewhat cheaper to construct per cubic yard. In any case the junction of the two straight lines of the face in the section should be connected by a curve, the radius of which may be made equal to the width of the top of the dam.

If the dam is trapezoidal in shape, Fig. 70, section *C*, the calculation is as follows:

$$W=m\left(\frac{th+bh}{2}\right);$$

$$P=\frac{wd^2}{2};$$

$$R=\tfrac{1}{2}\sqrt{m^2h^2(t+b)^2+w^2d^4}.$$

A general rule may be worked out for the length of t and b in terms of h, d and m; but it is very complicated and it is simpler to use the regular methods for calculating in each case the location of the center of gravity O, and the point where R intersects the base. If R intersects the middle third and does not exceed the crushing strength of the base, it is not necessary to calculate for any parts of this section extending from the crest to horizontal planes at intervals above the base.

In the case of high dams, when it is of financial moment to reduce the volume of masonry to a minimum, the weight of the wide crest must be taken into account. Including this weight in the calculation, the profile of the dam section becomes as shown in Fig. 71. The profile without taking the wide crest into account would increase the amount of masonry by about 5 per cent.

In Fig. 71 the dotted lines are loci of the points of application of the resultants when the dam is empty and full respectively. They are found by calculating these points and the necessary widths at horizontal planes taken at intervals of 5 feet, considering the entire portion above each plane as a dam; and connecting these points by the loci shown. The profiles of two other masonry dams are shown in broken lines, the Austin, Tex., being an overflow dam.

Water falling over a high dam with a straight, steep batter to the face will cause considerable shock to the foundation and ebullition of the water, tending to undermine the dam or loosen the bond of the masonry joints by jarring. To prevent this, when the dam is on soft rock or is more than 15 or 20 feet high, one plan is to curve the toe concave upward, as in the Holyoke dam, and also curve the front of the crest, giving the face the form of an ogee curve, which the water will slide along and thus produce no jar. In a number of dams the face is broken into steps which the water strikes in succession as it descends, thus

using up a large part of its energy and greatly reducing the force of the blow and the scouring velocity. Also this is cheaper

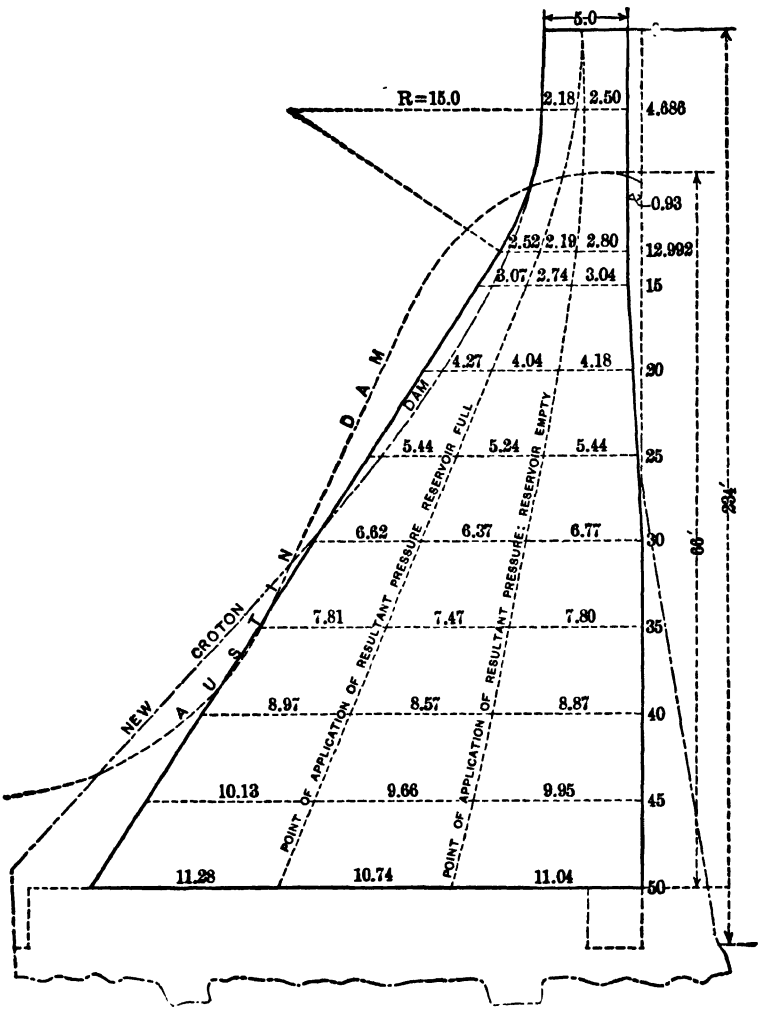


FIG. 71.—Practical Economic Profile of High Masonry Dam.

than cutting the face stone to an ogee curve, or than setting special curved forms for concrete; but stepping is not generally recommended for high concrete dams. Because of the same saving in cost, low dams are generally made trapezoidal in

section, the top width and height being decided upon, and the bottom width being made sufficient to insure the stability of the dam. With concrete dams, however, where exact and curved outlines are merely a matter of constructing forms, there is much less increase in cost in constructing to theoretical lines.

To provide the greatest amount of storage possible, a dam must be raised to a level with the highest permissible water line; but as a large amount of water at times passes over the dam or its spillway, either the water in the reservoir must at such times rise above this water line, or else it must ordinarily stand below it. Because of this, and also sometimes to store more water than was contemplated by the designer, temporary "flash-boards" are placed on the top of the overflow section, raising it from a few inches to 3 feet or more in some cases. The use of these is not recommended unless so designed that they will certainly break or open before the water rises above the high-water line.

If a dam be curved in plan, the masonry may act as an arch, the ends abutting against solid bed-rock in the sides of the valley. It is thought by some that the arch effect cannot come into play unless the section of the dam is too light to support itself by gravity, and yields under pressure, in which case the dam must slide bodily, must break bond along both vertical and horizontal planes, or must change form without any partial rupture. Very little is known of the law of stresses in such a dam; but that it can act as an arch is demonstrated by the Bear valley, Zola, and other dams, the former of which, if acting as a gravity dam only, would be destroyed long before the water had reached its crest. Since 1887 several curved dams 120 to 324 feet high have been built in the United States. The arch plan undoubtedly gives a margin of safety to a dam, even if it be designed with a gravity section.

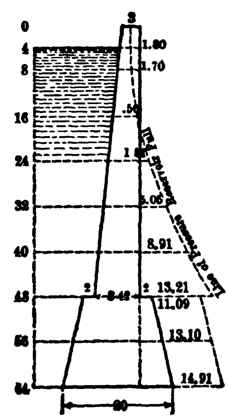


FIG 72.—Bear Valley Dam.

Recently dams of reinforced concrete have been designed

of the multi-arch form, the thrust of the several arches being taken by buttresses built on the stream bed below the dam. One at Gem Lake, Cal., consists of sixteen arches each 40 feet long, and a fractional arch at each end. The maximum elevation is 112 feet above foundation and 84 feet above the ground level; the maximum thickness of arch is 3.6 feet, and the arches are 1 foot thick at the crest. The arches make an angle of about 45° with the vertical. Waterproofing plaster was applied to the concrete with a cement gun.

More common are the cellular or hollow dams. These, as well as the multi-arch dams, rely almost entirely upon tensile and compressive strength and bonding to the foundation, and very little upon weight. The up-stream face of the dam is given an angle of about 45° with the horizontal, when (see Fig. 68) the resultant static pressure of the water cuts the base at the outer limit of the middle third, if the base ends vertically under the crest. If the base extend beyond this, the face of the dam may be made steeper. The dam consists essentially of a number of walls or buttresses set at right angles to the line of the dam, their up-stream edges connected by a continuous reinforced concrete slab. If it is an overflow dam, the buttresses are generally battered on the down-stream edge also, and these edges also are connected by a concrete slab to guide the overflow water; in which case the up-stream face can be made steeper than 45°. If the buttresses rest on solid rock no other foundation is necessary; but if they do not, a continuous concrete slab is carried under all the buttresses as a foundation, and cut-off walls at and near the up-stream edge are carried down to impervious material to prevent undermining.

In designing such dams, approved rules for reinforced concrete slabs are employed, the load being the hydrostatic pressure, and the dynamic pressures previously named. Temperature changes in length are provided for by expansion joints over the buttresses. Generally an opening is left through each buttress to give access to the inside of the dam for making inspections, and possibly minor repairs to prevent leakage. (Steel dams have been constructed on the same principle as the

cellular reinforced concrete. While cheaper to construct, they are less durable, and more expensive to maintain if rusting is to be prevented.)

CONSTRUCTING MASONRY DAMS

In beginning the construction of a masonry dam, every square foot of the foundation should be thoroughly cleaned and washed, and covered with a thick bed of mortar just before the masonry is laid upon it; and the spaces between the dam and the sides of the rock excavation—if any was made—should be cleaned out and filled with concrete.

The ends of the dam should be carried to bed-rock, if possible, and treated as was the bottom. If rock does not extend up to the level of the crest of the dam at its ends, these should be carried some distance into the banks to prevent leakage around them, or tight masonry river-walls should be carried for some distance up-stream from the dam. The banks just below the dam, if of earth, must be protected from wash by similar walls carried for a short distance down-stream from the dam.

If the dam forms a reservoir, or it is desired for any reason to draw off the water at any elevation lower than the crest, pipes or other conduits must generally be carried through the dam. These should never be given uninterrupted smooth outside surfaces where they pass through the masonry, but should be provided with flanges or other projections around their circumferences to prevent water from following along them through the wall. Probably the best plan is to place two or three wide flanges around the pipe near the upper face of the wall and bed these thoroughly in concrete, having first cleaned them and the pipe and given both a coat of rich Portland cement mortar. Other flanges should be placed at intervals of 10 to 20 feet if the dam exceed this thickness.

In some cases a tunnel is carried through the dam, open at the lower end, and terminating at the upper in a watertight well rising above the water surface, in which tunnel the conduits are laid. This is an excellent plan for large dams.

Masonry dams may be constructed of either rubble, cyclo-

pean, or concrete masonry; and the last either plain or reinforced. Brick masonry has been used, but is deficient in weight and in crushing strength and is never advisable if other material is available.

In building a rubble dam, care should be taken to break courses at every joint; the masonry being the most uncoursed rubble, except the faces, which are generally broken ashlar. Every stone and its bed must be perfectly clean and have a damp surface when it is laid, and all spaces must be absolutely filled with mortar or fine concrete. If a tight dam is to be built these precuations must be conscientiously observed, although it may be necessary to discharge half the masons during the first week to effect this. The dam should, so far as possible, be so carried up that the top of the finished masonry is at all times approximately horizontal, but very irregular in surface. Every stone should be set by derrick; and no dressing of stone should be done on the wall, as this is likely to disturb or jar stones already set. Sharp corners and angles on stones should be knocked off, as rounded stones bed in mortar more thoroughly than squared ones. Spalls are desirable in large joints; but these should be forced into mortar previously placed—*never place the spalls first and the mortar afterward.* The large stones will generally form 50 to 60 per cent of a well-built dam, the spalls 15 to 30 per cent, and the mortar 20 to 30 per cent.

Cyclopean masonry is a cross between rubble and concrete with "plums"; large stones being used, with joints 1 to 3 feet wide filled with concrete, the stones being bedded in concrete 1 to 2 feet thick. No large stones should be less than 6 inches apart at any point. The stones are allowed to project a few inches above any temporarily finished surface of concrete to furnish a bond with the next course.

Concrete has been used for most of the modern dams, sometimes faced with stone ashlar. Where the dam is of any considerable size a saving in cost is effected by embedding in the concrete large derrick stones called "plums." These should never come within several inches of touching each other, and should not constitute more than 30 per cent of that part of the

Table No. 40

DATA OF MASONRY DAMS

By Edward Wegmann

Dam.	Location.	Date of Construction.	Depth of Water, in Feet.	Total Height above Bed Rock, Feet.	Thickness at		Area of Profile, Square Feet.	Max. Pressure in Masonry, Tons per Square Foot.	Weight of Masonry per Cubic Foot.	Length at		Plan.
					Top, in Feet.	Base, in Feet.				Top, in Feet.	Base, in Feet.	
Puentes *	Spain	1791	153.5	164.2	35.7	144.3	16,349	8.12	.....	925	70	Polygonal
Zola	France	1843	119.8	123.0	19.0	41.8	3,645	8.12	.....	205	23	Curved, $R=158$
Furens	"	1866	164.0	170.6	9.9	161.0	10,712	6.82	.....	328	30	Curved, $R=828.4$
Ternay	"	1868	112.7	124.7	13.1	81.7	8,355	7.16	147.0	.	...	Curved, $R=1312.4$
Habra †	Algiers	1873	116.8	124.7	14.1	95.0	5,584	6.31	134.0	1476	.	Straight
Boyd's Corner	United States	1872	...	78.0	8.6	57.0	2,039	.....	.....	670	200	"
Gileppe	Belgium	1875	147.6	154.2	49.2	216.5	18,708	6.14	143.5	771	269	Curved, $R=1640.4$
Villar	Spain	1878	162.3	170.3	14.7	154.5	11,596	9.60	143.0	546	...	Curved, $R=440$
Bouzey ‡	France	1882	.....	75.5	13.1	45.9	1,961	11.26	.....	1545	...	Straight
Harniz	Algiers	1885	114.8	134.5	16.4	91.2	5,629	11.25	.....	532	131	"
Bridgeport	United States	1888	.....	40.0	....	...	.....	.....	.....	640	50	"
Vyrnwy	England	1882	.....	146.0	20.0	117.8	8,972	8.70	160.8	1350	...	"
San Mateo	United States	1889	.....	170.0	20.0	176.0	16,660	.....	.....	700	...	Curved, $R=637$
Sodom	"	1893	.....	98.0	12.0	53.0	.....	.....	.....	500	150	Straight
Tansa	India	1891	.....	118.0	12.0	100.0	.....	9.00	.....	8800	...	"
Bear Valley	United States	1884	60.0	64.0	3.2	20.0	537	.....	.....	450	...	Curved, $R=300$
Sweetwater	"	1888	90.0	98.0	12.0	46.0	2,347	.....	164.0	380	...	Curved, $R=222$

* Ruptured by weak foundation. †Destruction caused by poor material. ‡ Destroyed by sliding on poor foundation.

TABLE No. 40—*Continued*

Dam.	Location.	Date of Construction.	Depth of Water, in Feet.	Total Height above Bed Rock, Feet.	Thickness at Top, in Feet.	Thickness at Base, in Feet.	Area of Profile, Square Feet.	Max. Pressure in Masonry, Tons per Square Foot.	Weight of Masonry per Cubic Foot.	Length at Top, in Feet.	Length at Base, in Feet.	Plan.
Mouche	France	1890	95.0	101	11.5	..		6.70	134.0	1346	...	Straight
Austin, Tex., No. 1 *	United States	1892	60.0	67		.	...	..	.....	1275	..	"
Butte City	"	1892	.....	120	10.0	83.0	.	...	.....	350	..	Curved, $R=350$
Titicus	"	....	...	135	18.0	75.0	5,209	....	....	534	...	Straight
Lagrange	"	1890	.	125	24.0	90.0	..	...	.....	320	..	Curved
Hemmet	"	1895	.....	136	10.0	100.0	..	....	.....	...	...	Curved
New Croton	"	1907	150	297	22.0	206.0	...	...	.....	2168	...	Straight
Lake Cheesman	"	1904	..	232	18.0	176.0	...	....	..	....	...	Curved
Spier Falls	"	1905	80	154	..	.	....	.	....	1359	...	Straight
Boonton	"	1905	105	114	17.0	77.0	..	.	....	2150	..	"
Wachusett	"	1906	....	228	25.8	187.0	.	..	.....	1476	..	"
Roosevelt	"	1911	240	280	16.0	170.0	..	...	.....	1080	..	Curved
Pathfinder	"	1910	...	206	10.0	94.0	....	....	.....	425	...	"
Shoshone	"	1910	....	324	10.0	108.0	...	.....	.....	200	..	"
Cross River	"	1909	106	170	23.0	116.3	....	.....	...	772	...	Straight
Croton Falls	"	1911	97	173	23.0	127.7	....	..	.....	1100	...	"
Olive Bridge	"	1913	210	252	26.3	200.0	.....	...	...	1000	...	"
Assuan	Egypt	1911†	82	112	36.0	.....	.....	....	...	6200	...	"

* Destroyed by sliding on poor foundation. † Raised in 1911.

concrete lying inside of the outer 2 feet of the dam, and should not form any part of such outer 2 feet. The concrete ingredients should be carefully proportioned for maximum density. The consistency of the concrete should be quaking or slightly dryer—as dry as will permit the concrete to flow into all crevices with light tamping. Continuous placing without stopping day or night is desirable; but if the placing is interrupted at any one point, all the precautions of the best concrete construction should be taken to insure thorough bonding of old and new concrete.

It is common practice to built a concrete dam in sections separated by vertical joints extending from the top to bottom of dam. Alternate sections are built first, with vertical grooves left at each joint; and after these have set, the remaining sections are then built. This method of construction permits each section to be built by continuous work as a monolith. The joints also serve as expansion joints. The grooves first built are sometimes coated with tar to prevent their adhering to the tongues of the adjoining section. Also in some cases a steel plate is set extending from top to bottom of a joint and set with one half its width in each section to form a water-tight stop against leakage; or a copper plate is used, buckled between the two sections so that it can straighten out if the joint opens by contraction of the concrete.

Art. 67. Rock-fill Dams

In many parts of the Western United States, sites of proposed dams are at such a distance from any railroad and transportation to them is so difficult that a masonry dam becomes a very expensive structure. In many cases there is, however, plenty of rock at or near the surface, and narrow canyons are available for a dam site. In a few such locations dams have been constructed by simply depositing rough stone, as it was blasted out and with no dressing, in an embankment of the desired height, the stones being carefully arranged, smaller stones filling the interstices of the larger to give them a stable position and prevent after-settling. The front and back faces are generally

lined with dressed rubble, dry or in cement, or with quarry stone laid carefully by hand and well chinked with spalls. Such dams are not expected to be absolutely tight, and should never be used as weir-dams. They are not adapted to the construction of storage reservoirs, because of their porosity; but will generally silt up in time and become reasonably tight.

Since more or less water will find its way through a rock-fill dam, the foundation should be on rock, as any other material is liable to erosion which may prove fatal to the dam. Since the dam acts as an embankment without cohesion, the side slopes must not be steeper than the natural "angle of repose" of the material. This will generally require a slope of at least 1 : 1; but it may be made $\frac{1}{2}$ or even $\frac{1}{3}$ to 1 on the back when this is faced with a heavy, well-built wall. In construction, the material should be carried up in approximately horizontal layers, the middle being always kept a little lower than the faces; and the face-stone should be kept up to the level of the loose rock. The greatest care should be used to make the dam compact by the use of spalls and quarry-chips filling all crevices; but no earth should be used, except that clean sand might to advantage be sifted down into the crevices of stone already laid. The faces should be of one- and two-man stone, well bonded and chinked.

The back of a rock-fill dam is in some cases made more nearly water-tight by banking earth against it, as in the case of the Idaho Company's dam, in which the earth is 3 feet thick at the top and 20 feet at the bottom; and the Pecos Valley dams (see Fig. 73); or by wooden sheathing, as in the case of the Walnut Grove dam, in which two thicknesses of 3-inch planking were used, with tarred paper between. In one—the Lower Otay dam—it was attempted to make the dam water-tight by the use of a heart-wall or sheeting through the center of the dam, composed of No. 0 to No. 3 steel plates, imbedded in concrete 1 foot thick on each side. This construction was carried to the top of the dam, and to a masonry foundation on bed-rock, through its entire length. The loose rock, with a slope of $1\frac{1}{2}$ to 1, was carried down only to the ground-surface.

Still another style of rock-fill dam is the Castlewood dam, which has both faces of rubble masonry, the upper being 6 feet

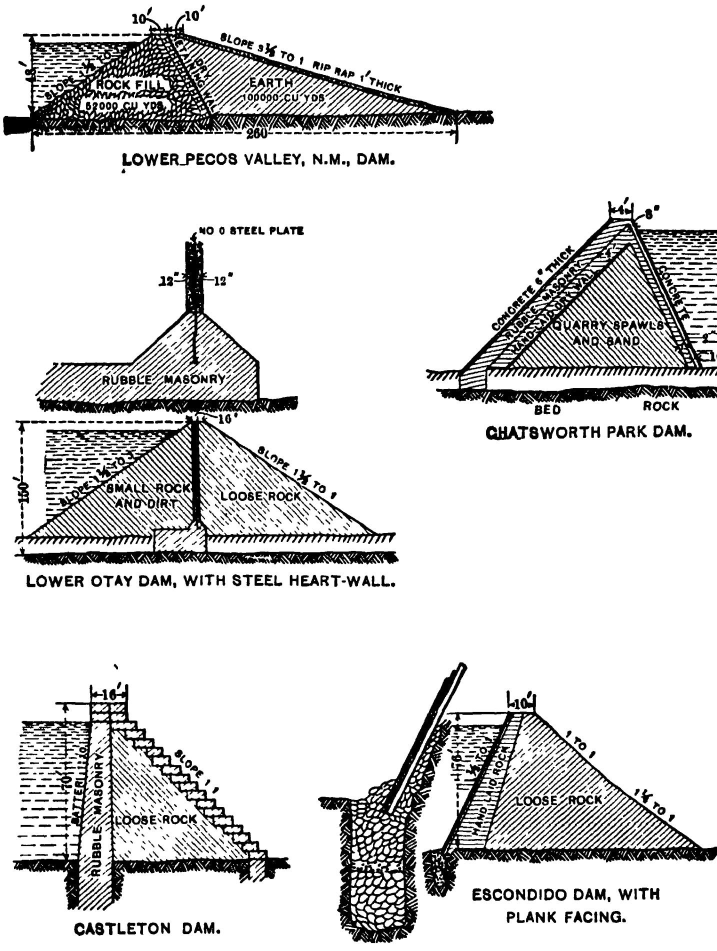


FIG. 73.—Rock Fill Dams.

thick on top and 12 feet on the bottom, and the lower from 5 to 7 feet in thickness, the heart being dry-laid rubble.

ART. 68. TIMBER DAMS

Timber dams cannot be considered as other than temporary structures, which must ultimately fail by the rotting

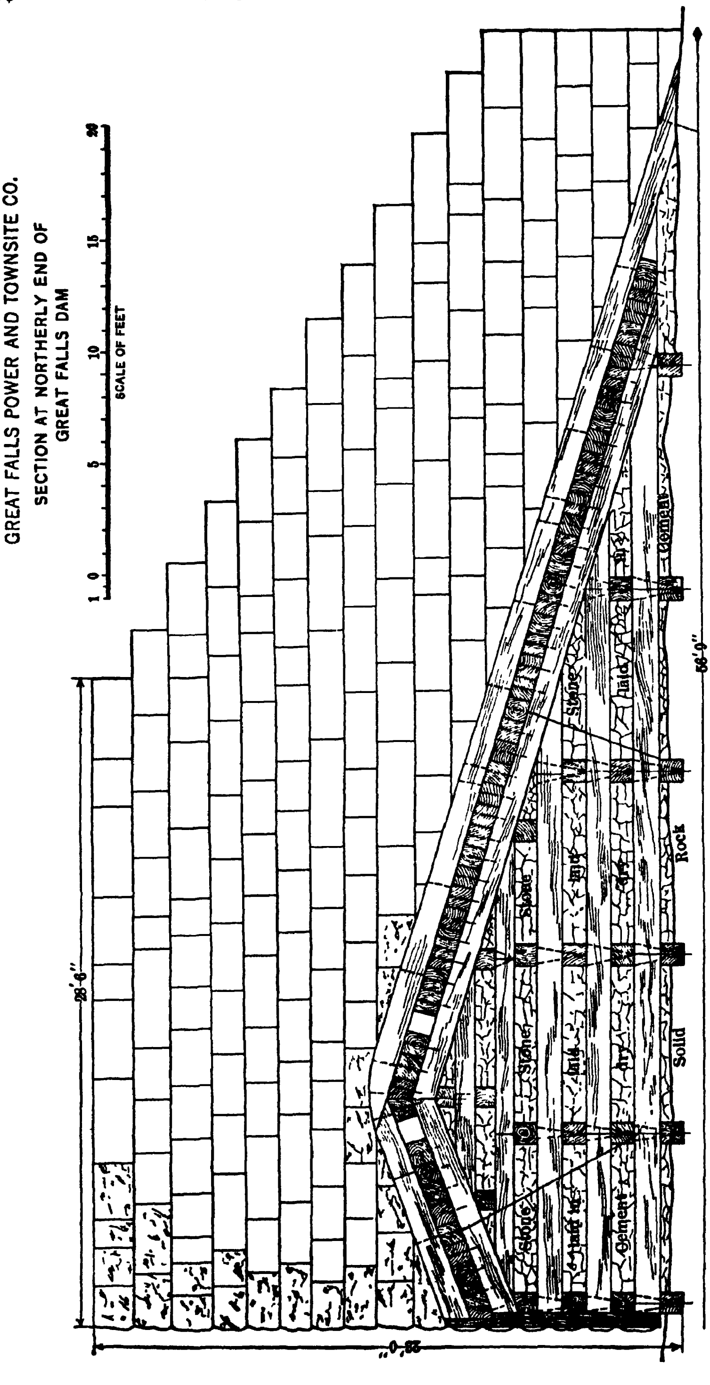


FIG. 74.—Great Falls Overflow Dam.

of the wood; and they are seldom very tight. Their general construction is that of cribwork weighted with stone, and faced with planking. They are placed upon rock or boulder and gravel foundations—sometimes upon clay or other firm soil. The chief or only advantage over masonry is in cost, which will ordinarily be but one-fifth to three-fifths as great for timber dams in a wooded country.

If the bottom is hard, cribs are weighted with stone and sunk directly upon it; but if soft, rip-rap or loose stone is placed over the bottom at the site of the dam and allowed to settle as far as possible, extending for some distance above and below the dam, and when leveled off the cribs are sunk upon this. The timber is usually 12 by 12 inches or larger, drift-bolted together.

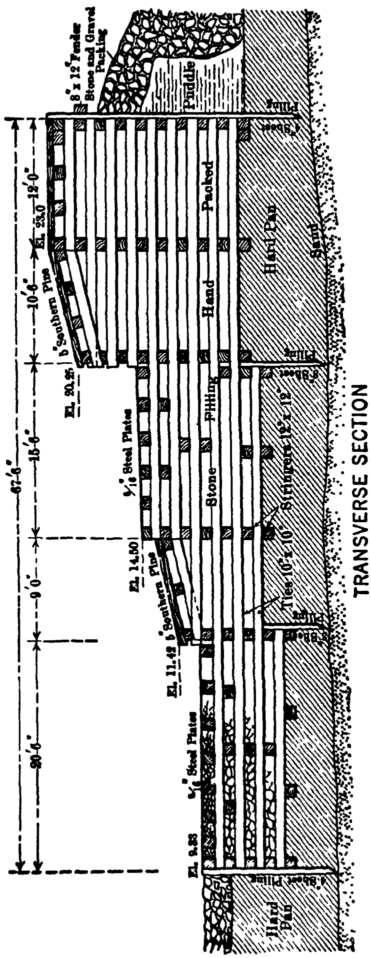


FIG. 75.—Sewall Falls Dam.

Figs. 74, 75 and 76 show three types of timber dam construction; the first an overflow 14 feet high on solid rock, with masonry river walls at the ends; the second 13.6 feet high and 407 feet long on hardpan, protected above and below with rip-rap, with four cut-off walls of sheet-piling and great width of base; and the last anchored to rock by fish-tail bolts.

ART. 69. EARTH EMBANKMENTS

Earth has probably been used for more dams and embankments in connection with water-works than all other substances combined, and serves this purpose admirably if properly designed and constructed; but it can never, under any condition, be used for an overflow dam, but a spillway or waste weir must be constructed in connection with it. Many earth dams have failed, but in the majority of cases this has been because a too-small spillway has forced water to flow over the dam.

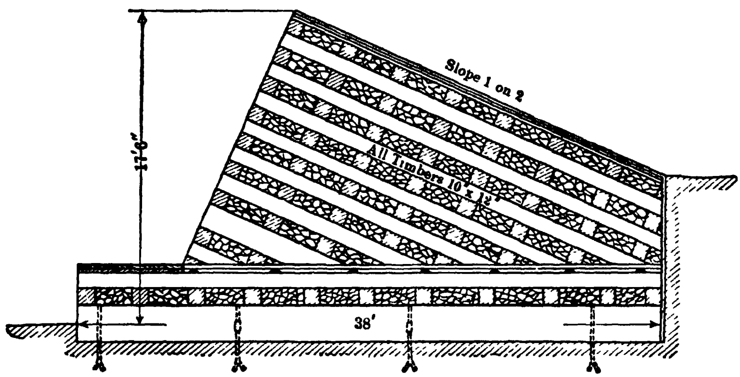


FIG. 76.—Cross-section of Bear River Dam.

An earthen dam is never absolutely tight, a slow seepage continually carrying a small amount of water through it, and if this becomes sufficient in amount or velocity to cause any movement of earth particles, the destruction of the dam is threatened. In addition to good foundation and ample spillway, the great requisites of a safe earth dam are that it shall be so compact that no water can pass through it as a stream, however fine; and that there shall be no continuous smooth surface, as of pipe or masonry, through it which the water may follow.

The pressure due to the head tends to force water in a reservoir through the earth embankment, and this is resisted by the friction in the pores. The finer the pores and the greater the distance through which the water must pass, the greater the friction and the less the seepage. Hence the bottom of an

earth dam should be much thicker than the top. At a depth of 75 or 80 feet the saturation of the earth reaches an amount which some engineers would fix as a limit, although at least eleven earth dams have been built in this country more than 100 feet high, and the U. S. Reclamation Service has built one—the Tieton dam—232 feet high with a concrete core wall.

The outer or down-stream half of an earth dam plays practically no part in effecting water-tightness unless the dam leaks very badly, but must furnish most of the stability; while the up-stream half furnishes the imperviousness. Anything approaching saturation in the down-stream half is therefore to be avoided, and some engineers place drains under this part of an embankment to remove any water that may reach it. If soil that makes impervious embankment is not available for the entire dam, it should be used for the half next to the water, and the coarser material for the lower or outer half. Even loose rock carefully placed has been used for this.

The faces of an earth dam must be at least as flat as the angle of repose of the material used: and this when the material is moist or wet. This angle for different soils is approximately as follows:

TABLE No. 41

ANGLE OF REPOSE OF SOIL

Kind of Soil.	Angle of Repose, Degrees	Equivalent Slope.	Coefficient of Friction.	Weight, Lbs. per Cu. Ft.
Earth, dry	40	1.2 to 1	0.84	90
Earth, moist	45	1 to 1	1.00	95
Earth, very wet	32	1.6 to 1	0.62	115
Sand, dry	35	1.4 to 1	0.70	100
Sand, moist	40	1.2 to 1	0.84	110
Sand, very wet	30	1.7 to 1	0.58	125
Gravel, round	30	1.7 to 1	0.58	100
Gravel, sharp	40	1.2 to 1	0.84	110

From this it would appear that an embankment of earth should be given a slope of at least $1\frac{1}{2}$ to 1, and one of sand $1\frac{3}{4}$ to 1. To insure stability, however, and reduce percolation, and to prevent undue wash of the banks by rain, it is

customary and advisable to give an inside slope of 2 to 1 and an outside of 2 or $2\frac{1}{2}$ to 1. If the dam is quite high, a still greater thickness at the bottom than these slopes would give may be desirable to prevent percolation; also there is danger that the entire inside slope may, on account of its great mass, slip down the slope. On the outside of a high bank the rainfall upon it gradually collects in rivulets and tends to wash the soil. To prevent these objectionable results an offset or berm 5 to 10 feet wide is generally made about half-way up each face of high embankments.

If the slope is less than $2\frac{1}{2}$ to 1 at the top of an embankment that is more than 40 or 50 feet high, it should be given this slope below that distance from the top on the inside, and a slope of 3 or 4 to 1 on the outside.

The top of the embankment should be at least 6 or 8 feet wide, and if the dam is more than 20 or 30 feet high it should be 10 to 30 feet wide. It is generally made wide enough to act as a driveway. The embankment should be carried at least a foot higher than the highest waves coincident with the highest water—about 3 to 6 feet above high water, depending upon the length of the reservoir. The elevation above high water should also be at least the depth to which frost reaches.

On the outside berm, gutters should be placed to catch the drainage from the upper slope, and drains to lead it to the toe of the dam.

In the above the embankment has been considered to be of uniform homogeneous material. In many embankments, however, a core-wall is carried through the middle, constructed of masonry or water-tight puddle; and in others a water-tight lining of puddle or concrete is placed on the face of the embankment. In the latter case the bank is supposed to offer stability only and not to be water-tight; in the former that portion of the bank outside the core-wall is for stability only, that inside assists in preventing percolation. When a core-wall is provided the outer slope need not generally be flatter than 2 to 1.

Some engineers object to the use of a core-wall, claiming that, since it and the remainder of the dam are not homogeneous,

cracks will open between during settling, to the weakening of the dam. It is undoubtedly true that a comparatively water-tight dam may be made of uniform material throughout, and one perfectly safe from all but the boring of animals, and small reservoirs may very properly be so built. But for large dams or those whose rupture would be attended with great damage or loss of life, the author would wish to use a core-wall; and he would not recommend a puddle core-wall except to secure tightness where there could be obtained only a small amount of clay or other puddling material. The masonry core-wall may be of stone or concrete; preferably the latter, because this can be made more water-tight, and smoother so that the earth can move freely over it in settling and thus compact more

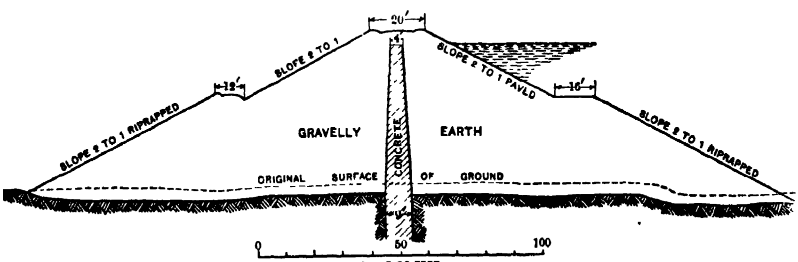


FIG. 77.—Oak Ridge Reservoir Dam.

solidly. It need not be heavy enough to resist any considerable pressure, this being sustained by the embankment.

Not only should a dam be tight itself, but it should be upon a water-tight foundation—rock, hardpan, or a thick clay stratum. If water penetrate under a dam through a soil subject to erosion, a cavity will be created which will sooner or later cause the destruction of the dam. A seamy rock may permit the passage of water under or around a dam without endangering its stability; but this is undesirable because of the loss of water, and the dam may be made useless thereby, as was a dam at Roanoke, Va. In the latter case the crevices should be sought out and filled with concrete or grout; or it may be necessary to build a masonry wall as a lining to the seamy sides of a reservoir.

If the rock or hardpan at the bottom of the valley is covered with porous material—as is generally the case—this should be removed and the embankment founded on only solid, continuous impervious material. If the pervious surface material is quite deep and the dam high, this may require

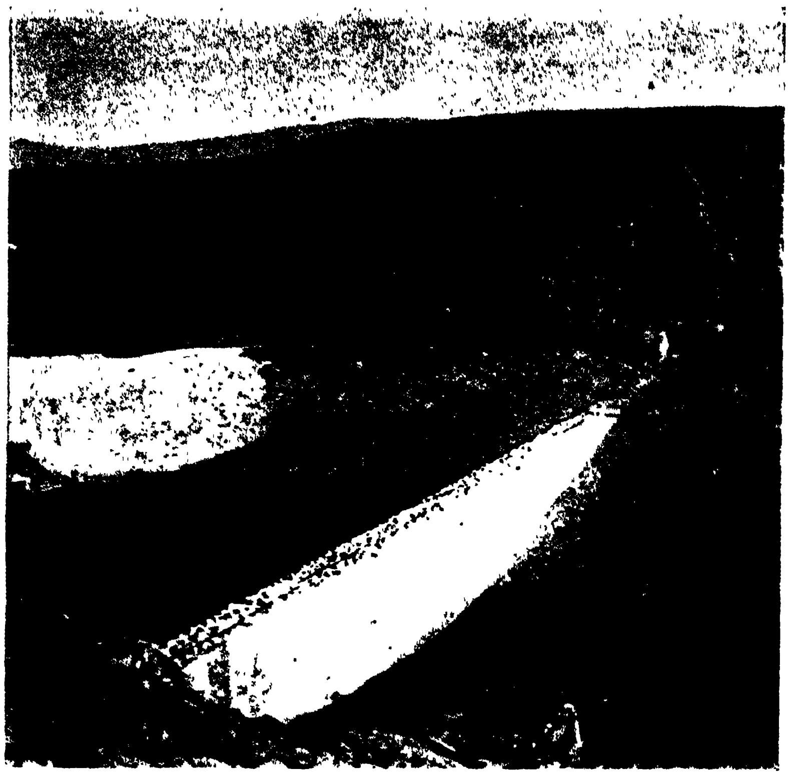


FIG. 78.—Concrete Core Wall of Fairmont Dam, Los Angeles Water Supply (Unfinished).

the excavation of an enormous amount of material. Thus a dam 40 feet high, top 10 feet wide, side slopes 2 and $2\frac{1}{2}$ to 1, will have a bottom width of 190 feet and if 10 feet of soil be removed this will require the excavation of 70 cubic yards per running foot of bank. To avoid the great expense and delay occasioned by this, a puddle or masonry cut-off wall is generally carried to the impervious stratum, and continued

as a core-wall, if such be used, or stopped a few feet above the base of the embankment if no core-wall be used; or, if lining be given to the dam, the cut-off wall is placed under and joined to the foot of this lining (see Fig. 80). The Oak Ridge reservoir dam of the East Jersey Water Company (Fig. 77) is a good illustration of a dam with a concrete core (or heart) and cut-off wall.

Not only the bottom but also the ends of a dam must make an impervious union with the natural soil. For this purpose the core-wall should be carried to bed-rock at the ends of the dam, if the rock rise there above the water level; or for some distance into the bank, if they do not. Or, if there be no core wall, the embankment should be extended until it reaches an impervious material, or for a distance into the sides of the valley equal to, say, 10 feet plus half its height if no impervious material be found short of this.

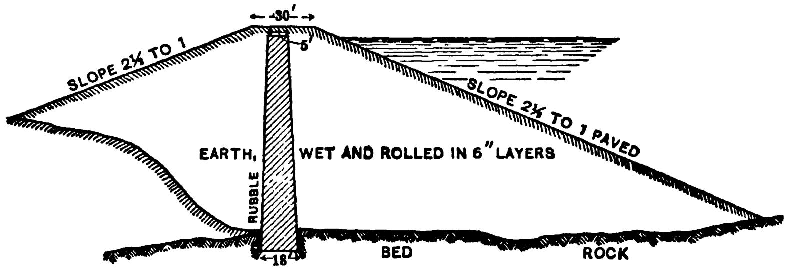


FIG. 79.—Dam of Reservoir M, New York Water Supply.

Almost every character of soil has been used for embankments, and there are few from which a reasonably good embankment cannot be made. Probably the best material is a sandy loam with a small amount of clay intermixed, and the worst is micaceous clay. Contrary to a widespread opinion, clay is not a good material for embankments in any but small quantities. A tough clay it is difficult—almost impossible—to get into a compact, homogeneous mass; if wet, it cracks open on drying out; and, most serious of all, if the smallest trickle of water once finds passage through it, a few minutes suffice to enlarge this into a break, so rapidly does it dissolve

in running water. An embankment of clay is often tighter at first than one of any other material, but the danger of its rupture increases with age. A bank containing a large amount of gravel or sand, on the other hand, may leak at first, but becomes tighter and stronger with age. A reservoir embankment 16 feet high, built of fine sand, with slopes of $1\frac{1}{2}$ to 1 covered with loam, has been found to suffer no appreciable leakage. Gravel as taken from the bank, consisting of stones, sand, and sufficient clay or loam free from vegetable matter to fill all interstices, makes an excellent embankment material. "Gravel capable of being puddled will do anything that clay was ever used for in

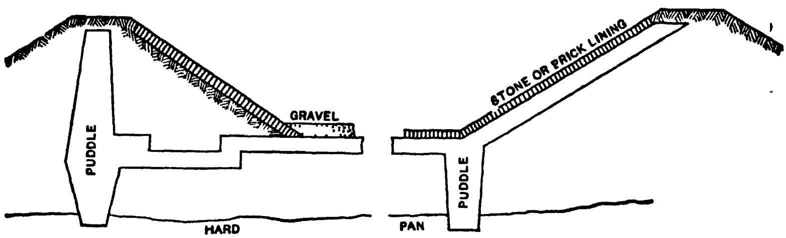


FIG. 80.—Puddle-lined Reservoirs.

water-works practice, and will do it better" (Clemens Herschel). Hardpan, the most impervious soil-mixture found in nature, contains a large proportion of gravel.

If the following materials are obtainable, they may be used to advantage in the proportion given, to make a strong embankment; although up to $\frac{1}{4}$ of clay, if thoroughly mixed, may make a tighter one. The amounts here given will make 1 cubic yard:

	Cubic Yard.
Coarse gravel	$\frac{3}{4}$
Fine gravel or coarse sand	$\frac{1}{4}$
Fine sand	$\frac{1}{8}$
Clay or loam	$\frac{1}{8}$

These materials should be thoroughly mixed, the clay being broken up or cross-cut into fine pieces, made slightly damp, spread in 6- to 9-inch layers, and thoroughly rammed, or rolled with grooved rollers. If the gravel and sand retain

their natural moisture no water should be used in the material. Embankment material is seldom wet too little, often too much. Natural hardpan, if broken up and rolled dry, will make a tight dam if water be admitted behind it slowly, the clay taking up water by capillary attraction and swelling. Puddle should never be made wet enough to quake, as it would then be porous upon draining or drying out.

During construction, all surfaces coming into contact should be rough. The ground upon which the dam is built should be stripped of all the soil which is porous or contains roots or other vegetable matter, and then plowed or spaded up, that the new and old material may unite; also the top surface of each layer of the embankment should be rough when more material is placed upon it, and the rollers used should be grooved and heavy—steam rollers where practicable. An earth embankment should never be placed upon rock, unless a heavy concrete core wall be used and carried down 3 feet or more into the bed-rock, as a water-tight joint cannot be made between earth and a smooth rock-bed.

In building up the embankment, the center should be kept somewhat lower than the faces; and these should extend 1 or 2 feet beyond the line of the finished bank and afterward be trimmed off, since the edge of a bank cannot be properly compacted. No large stones, nor any sticks, roots, or other matter which can decay, should be permitted in an embankment. If the material needs wetting, this should be done before the fresh layer is put on, rather than after; the bond between the fresh and old material being thus made more thorough, and the material being less liable to cling to the roller.

Nothing should extend through the dam when this can be avoided, and any pipe or other conduit which must pass through it should be furnished with a number of flanges and other projections, or, if of masonry, should be left as rough as possible; and the best of puddle, mixed and rammed with the greatest care, should surround it. Where possible it is ordinarily better to place the spillway or the overflow pipe (which latter may be used if the reservoir is small and does not receive

TABLE No. 42

DATA OF EARTH DAMS

Name.	Location.	Greatest Height.	Length on Crest.	Width of Crest.	Slope of Upper Face.	Slope of Lower Face.	Heart-wall.	Remarks.
Dudley Brook........	Windsor, Vt...........	31	170	4	1½ to 1	1½ to 1	None............	Clay and gravel; masonry spillway, no leakage.
Salisbury Brook......	Brockton, Mass.........	23.2	1524	16	2 to 1	1½ to 1	Concrete, 3′ to 1½′.	Clayey gravel; masonry spillway, no leakage.
Stony Brook.........	Cambridge, Mass......	25	706	20	1½ & 3 to 1	2 to 1	Masonry, 8′ to 3′..	Gravel; leaks around end.
Acushnet river.......	New Bedford, Mass....	.....	650	20	2 to 1	2 to 1	Puddle, 12′ to 4′..	Fine gravel, coarse and loamy sand.
Waterville.	New York............	17	240	10	2 to 1	2 to 1	Masonry, 3′ to 1½′.	Distributing reservoir.
Compton Hill........	St. Louis, Mo.........	26	.....	20	1½ to 1	2 to 1	Puddle, 7′ to 4′...	Puddled bottom; clayey loam. Storage reservoir.
Reservoir "M"......	Titicus river, N. Y.....	100	1000	30	2½ to 1	2½ to 1	Rubble, 18′ to 5′...	
Oak Ridge..........	Pequannock river, N. J.	60	620	20	2 to 1	2 to 1	Concrete, 9′ to 4′..	Overflow detached from dam.
Honey Lake Valley ..	California.....	90	960	20	3 to 1	2 to 1	Puddle, 25′ to 10′.	Crest 6′ about high water.
Dam No. 5..........	Boston, Mass..........	70	1865	14	2 to 1	2 to 1	Concrete, 10′ to 2′.	Central masonry waster weir.
Pilarcitos.......... ..	California.............	95	650	26	2¾ to 1	2 to 1	Puddle, 20′.......	Heart-wall 46 ft. below base of dam
San Andreas	California.............	95	650	25	3½ to 1	3 to 1	Puddle, 20′..	Heart-wall 48 ft. below base of dam
Crystal Springs......	California.............	50	560	30	3½ to 1	3 to 1	Puddle, 25′.......	Heart-wall 98 ft. below base of dam
San Leandro.........	California.............	125	600	28	3 to 1	2½ to 1	Puddle, 50′ to 40′..	Heart-wall 40 ft. below base of dam
Cedar Grove.........	New Jersey...........	55	...	18	3 to 1	2 to 1	Concrete.........	Bank of red clayey loam.
Bog Brook.	New York............	60	..	25	2 to 1	2½ to 1	Masonry, 10′ to 6′.	Berm on down-stream face.

drainage direct) at one end of the dam on the natural soil or the overflow pipe and the conduit may be carried in a tunnel which passes through the dam and terminates as its upper end in a tight inlet tower, as in the case of a masonry dam. Such a tunnel should in every case be carried down to bed-rock throughout its length. If a pipe or conduit be carried through the embankment it should rest, not on masonry piers (a construction too often adopted, to the endangering of the pipe by breaking between supports)—but upon (or better still, in) bed-rock or hardpan, or a continuous foundation of the roughest possible masonry in cement mortar carried down to rock or other firm bottom, and provided with projecting or cut-off walls to prevent water from following the masonry through the dam. Next to insufficient spillways, improperly built conduits through embankments have been the cause of the greatest number of ruptures in earth dams.

Where there is a masonry waste weir in the center of the dam, it is an excellent plan to carry all pipes or conduits through this, as was done at No. 5 of the Boston water works.

Table No. 42 gives data concerning a few earth dams as illustrations of general practice.

Art. 70. Hydraulic-fill Dam Construction

At least twenty large earth dams have been built in California and other Western states, and five in Ohio by the Miami Conservancy District, by what is known as the hydraulic method, or sluicing. The highest is believed to be that at Terrace, Colo., 210 feet maximum height and 605 feet long, and perhaps the smallest is at Tyler, Tex., 32 feet maximum height and 575 feet long. The Englewood dam, in Ohio, is 4660 feet long and 108 feet high. Several dams on the Los Angeles aqueduct were built by this method in 1914–1916.

In this method of construction the earth, gravel, etc., are delivered onto the dam by water, which transports the material through flumes or sometimes steel pipes. The middle of the dam is kept low and the material delivered near the edges, which

causes the coarser material to be deposited there and the finer to be deposited in the center of the dam by the water, which pools there. In most cases the soil is excavated by turning onto a bank jets of water from nozzles with 2- to 4-inch openings; the water, carrying the soil, flowing into a flume that carries it by gravity to the dam. In some cases the material is hauled by teams to a point near to the dam but higher, dumped here, and sluiced from here to the dam.

In the case of the Miami valley dams, selected materials were excavated in borrow pits, hauled by train to a sump, there mixed in the desired proportions and with water, and the semi-fluid mixture pumped by centrifugal dredge machines several

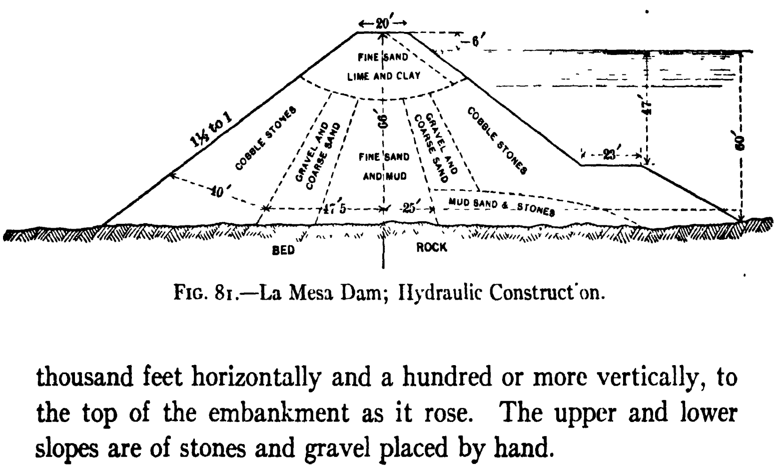


FIG. 81.—La Mesa Dam; Hydraulic Construction.

thousand feet horizontally and a hundred or more vertically, to the top of the embankment as it rose. The upper and lower slopes are of stones and gravel placed by hand.

From experience on the Los Angeles aqueduct dams it was concluded that for successful hydraulicking, there should be used at least 10 to 15 cubic feet of water per second, under a pressure at the nozzles of 100 pounds or more, and that the sluice must have a grade of at least 4 or 5 per cent to the dam (grades as high as 9 per cent have been used). Also that by the method described of getting the finer material in the center of the dam, soil with less than 35 per cent clay and other fine material can be used; which amount of fine material uniformly distributed throughout the dam would not make a tight embankment, without a core wall. On this work there were used 2-inch nozzles, mounted

on iron tripods with swivels, connected by 4-inch canvas hose to water mains.

In the Miami work the material was delivered through a 15-inch pipe of high-carbon manganese-steel, $\frac{11}{64}$ inch thick.

Art. 71. Reservoir Lining

Storage reservoirs are seldom less than an eighth of a square mile in area, and it would ordinarily be impracticable to line or cover these; and lining is not often necessary to secure tightness, since the reservoir is in most cases formed by a dam across a narrow valley, which dam, or its core-wall, can be carried as a cut-off wall to rock or clay on both bottom and sides.

A distributing reservoir, however, is more frequently constructed on a mountain side or top, and is built largely or wholly by excavation and embankment. This construction will in many cases produce a reservoir which is porous on the sides of the excavation; and on the bottom also, if rock or hardpan is not reached. Such a reservoir will require either that a core-wall carried down to clay or rock entirely surround the excavation, or that the reservoir be lined with some impervious material.

When rock occurs over the entire bottom of the reservoir or a considerable portion of it, it is practically impossible to render the reservoir tight by any material other than masonry, either as a lining or core-wall. Since the latter would call for a trench excavated to rock entirely around the reservoir, the masonry is in most if not all cases placed as a lining. If the excavation extends for any distance into rock, the face of the cut is frequently made vertical; and the earth should be faced with a water-tight lining or retaining-wall, since it is apt to slip on the rock-surface if water reaches such surface. Such a construction is shown in Fig. 82, a section of the Manchester, N. H., high-service reservoir. The bottom may be left unlined if the rock has no seams and is perfectly smooth; but it will generally be advisable to give it a lining of concrete—from 6 to 12 inches if to prevent leakage; otherwise, sufficient to level

off the surface and render easier the drawing off of all the water and the cleaning of the reservoir. The retaining wall should be practically water-tight; but is generally backed with earth to add to the imperviousness and stability, and should therefore be capable of retaining this without overturning when the reservoir is empty.

When rock forms the bottom of the reservoir but is not excavated at all, the earth may be given a slope of 1½ to 1 if

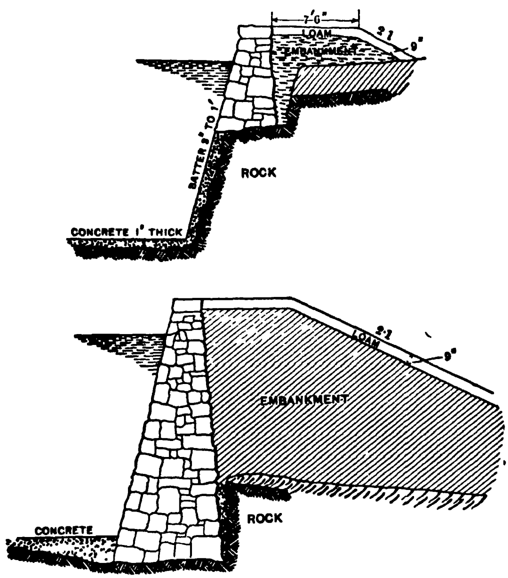


FIG. 82.—Reservoir in Rock Excavation; Manchester, N. H.

only 8 or 10 feet deep, but should be 2 to 1 if 15 or more feet deep. The use of clay puddle for a lining to such a reservoir might be adopted, but cannot be recommended as always satisfactory, concrete being a safer material for both bottom and sides. Embankment which is to be lined should be compacted with particular care, in order that it may not settle and crack the lining.

A reservoir entirely in earth may be lined with either concrete or clay. If the former be used, the bottom should be thoroughly rolled with heavy rollers to compact it; a layer

of gravel spread before rolling will be of assistance in securing this condition. If the bottom be of clay, this treatment will often make an artificial hardpan which will be as good as a clay puddle. If hardpan be reached in the bottom, it will ordinarily be necessary merely to level this off and roll it with a heavy flat roller. If no clay or hardpan be reached, a puddle lining may be used. This may be mixed in proportions of clay, 1 part; sand and fine gravel, 1 part; and coarse gravel, 2 parts. This should be put on in 4-inch layers and thoroughly rolled, the total thickness being 1 to 3 feet, depending upon the depth of water. This should be protected in some way from penetration by fish, etc., wash of water, and injury during cleaning of the reservoir. Six inches or so of gravel, with dry stone lining on the slopes, is often used for this purpose; but a better lining is made of 4 to 8 inches of concrete.

Clay lining is now seldom used for embankments, because the lining is apt to slip and to be loosened by frost, to crack open if alternately wet and dried, and to break if a settlement occurs in the bank; and because the same amount of puddle can generally be used to better advantage as a heart wall. A small reservoir on porous soil may require to be lined throughout, and in this case a puddle bottom lining may be used and continued either as a slope lining or as a heart wall. (See Fig. 80, page 336.) Clay lining should always be protected from injury; that on slopes by masonry.

Concrete is the material now commonly used for lining reservoirs. It is made from 6 to 9 inches thick, is sometimes (but not often) reinforced, and is generally made in blocks from 10 to 25 feet square, with expansion joints between them. On side slopes each block may extend the entire height of the slope, the only joints being the vertical ones. These joints may extend vertically through the lining and be filled with bituminous material such as is used for expansion joints in pavements (only "poured" joints should be used—not the solid expansion joint strips). In several cases a half-lap joint between blocks has been used, the upper face of the lower half of the lap being painted with asphalt or tar before the next block of con-

crete with the upper half of the lap is placed. Or a strip or base of concrete about 8 inches wide is placed under each joint, with its top surface painted with tar, the joint coming over the middle line of the strip. These joint constructions are of course to permit temperature movements without leakage.

The concrete used should contain no stones larger than 1-inch. *Clean* gravel probably makes tighter concrete than broken stone. The mixture should be rich and the aggregate proportioned to secure maximum density. It should be just wet enough to require light tamping to compact it. On the slopes it may not be possible to make it this wet, as it will slide down the slope, but it should be made as wet as it can be and still retain its shape; and must be tamped enough to make it thoroughly compact. It is desirable to float all surfaces down to a "sidewalk" finish. The concrete is sometimes given a waterproof wash. Two or three washes of cement grout applied with a brush to the concrete after it has set is advantageous, especially if the surface is at all porous.

Asphalt has been used as a lining in a number of western and a few eastern reservoirs, being placed upon a concrete lining or directly upon the earth embankment. But its use is less common that it was about 1890 to 1900. It is insoluble in water, giving it no taste or color; will yield with a slightly settling bank without cracking; is easily repaired; but it flows in the sun above water and does not retain its position well unless placed on concrete, in which case it is not necessary.

Art. 72. Covered Reservoirs

An impounding reservoir is of such size as to render a covering impracticable, and only small ones like distributing or clear-water reservoirs have been covered. Covering is most frequently desired when the supply is from ground-water, which is so often given an objectionable taste by the presence of algæ. By using covered reservoirs, the water is delivered to the consumer without having been at any time exposed to the light, without which very few algæ can exist. A covering is also desir-

able to exclude dust and other atmospheric impurities and to prevent malicious pollution; also to protect the water from heat and maintain a uniform temperature and to prevent loss by evaporation. A covered reservoir should be ventilated.

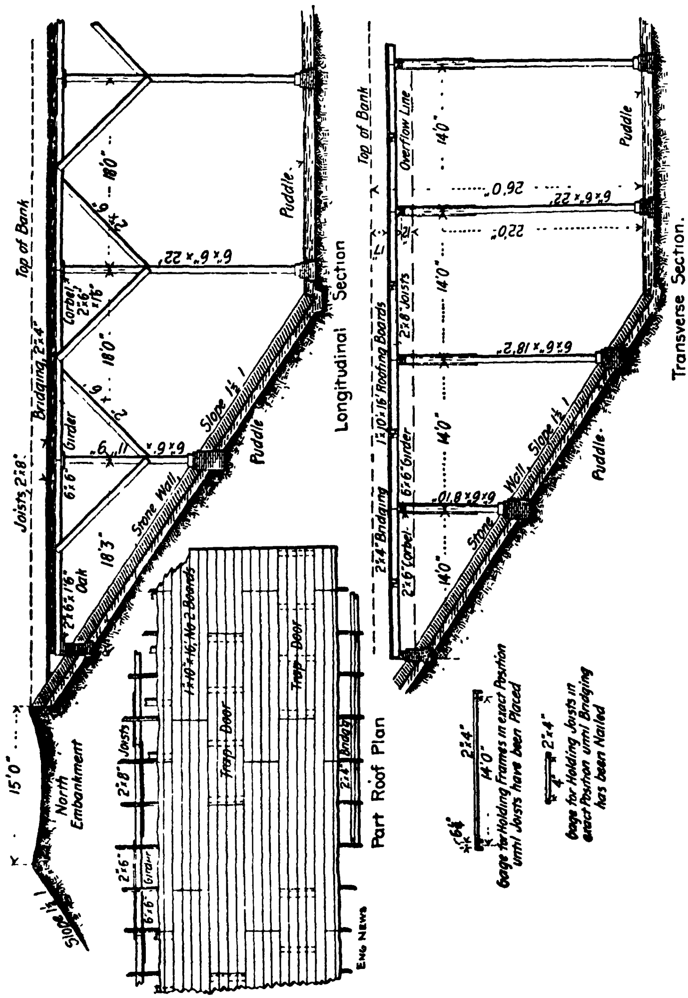
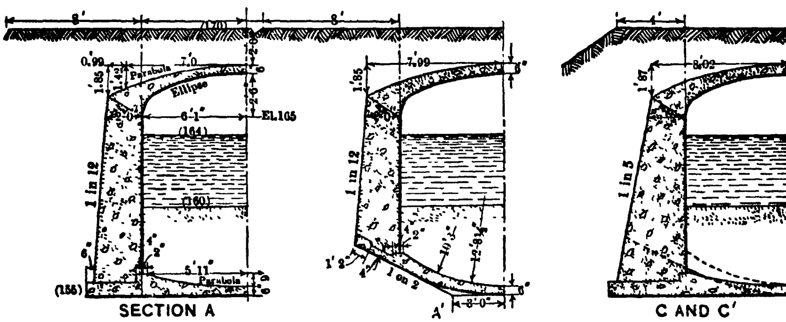


FIG. 83.—Covered Reservoir at Quincy, Ill.

The covering may be of timber, metal, or masonry. The first is generally the cheapest but least satisfactory. It is generally constructed by resting horizontal beams upon timber or iron posts, or brick piers, the beams being spaced equally

over the entire area of the reservoir and covered with plank. For small reservoirs but 40 or 50 feet in diameter, an ordinary circular wooden roof has been used, covered with tin or slate. For somewhat larger reservoirs, steel roof construction has been used. This, however, offers little protection from the sun's heat. At Quincy, Ill., a reservoir 415 by 317 feet was covered



SECTIONS OF WALLS USED FOR DIFFERENT WIDTHS OF EMBANKMENTS.
DEEP FOUNDATIONS USED IN SHALLOW CUT OR ON FILL. RAISED FOUNDATIONS (DENOTED BY PRIMED LETTERS) USED IN DEEP CUT.

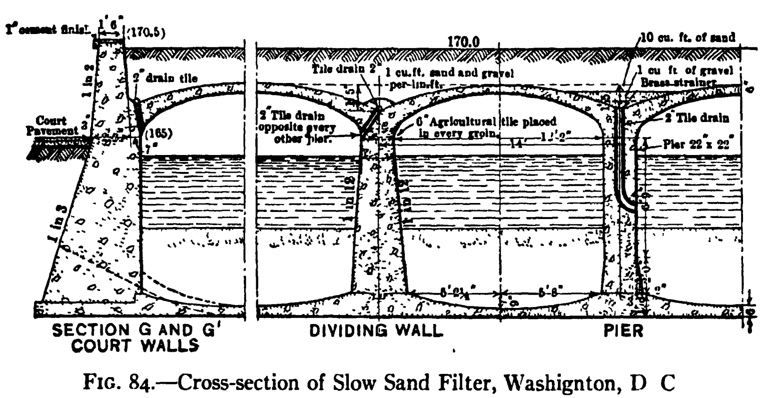


FIG. 84.—Cross-section of Slow Sand Filter, Washignton, D C

with timber by the first method (see Fig. 83). At Pasadena, Cal., five reservoirs were covered with timber roofs; the largest, 330 by 540 feet, using 2-inch iron pipe for posts and 4 by 10-inch by 18-foot girders. No snow falls in this climate; but the weight of this must be considered in designing roofs in Northern localities.

Concrete groined arches supported by concrete posts have for several years been the most common construction; this being practically the same as that used for slow sand filters. The roof is generally covered with 1 to 3 feet of earth to protect it from accident and the heat of the sun, one disadvantage of the latter being the temperature expansion and contraction of the concrete. Earth covering also improves the appearance of the structure. The piers rest upon inverted groined arches unless there is solid rock at the bottom of the reservoir. Standard

FIG. 85.—Interior of Covered Reservoir, Baltimore Filter Plant.

practice is to make the thickness of the piers 12 per cent of the span on centers, and at least one-twelfth the height, and the span from 12 to 18 feet. The thickness of the outside walls is about 12 per cent of their height at the top and 16 per cent at the bottom, and they are reinforced and calculated as beams supported at top and bottom and resisting the earth pressure. The roof is generally made 6 inches thick at the crown, and the thickness at the piers calculated by assuming joints along the diagonals. The rise of the intrados is about 20 to 25 per cent of the clear span, and that of the extrados about one-half the sum of this and the crown thickness. Recently a few reservoirs have been roofed with flat slabs of reinforced concrete supported by reinforced concrete beams.

CHAPTER XIV

DESIGNING

Art. 73. Collecting the Data

The problem presented to the designing engineer is generally: Given a certain city to be supplied; to decide upon a source of supply which will meet the requirements as to both quality and quantity, to develop this source properly, conduct the water to the point of utilization, and arrange for its distribution to meet the various requirements demanded of the system.

The first problem presented is to forecast the population and the amount required to provide the domestic, manufacturing and the commercial consumption during each decade for the next fifty years or more. The method of doing this is described in Chapter II.

A map of the city should be obtained or prepared showing curb as well as property lines, and the location as far as possible of all underground structures within the street limits, gas and other pipes, fire wells, subcellars or vaults, etc. A convenient scale for such map is 200 feet to 1 inch, but if the size of the city is such that this scale would necessitate the use of paper more than 3 feet wide it may be better to use a scale of 250 or even 300 feet to a 1 inch. It is inadvisable to use a smaller scale than this, and if the resulting map is still too large for the paper it may be necessary to spread it over two or more sheets, when the use of the 200-foot scale is advisable. Extreme accuracy in this map is not necessary for the planning of the system, but for preparing the estimate of cost it is desirable that the lengths of pipe may be scaled from the map with tolerable accuracy. It is advisable to limit the possible error in lineal dimensions to 0.5 per cent.

A topographical survey of the city can be used to advantage, but the benefits accruing do not ordinarily warrant incur-

ring the expense for this purpose alone; and a few levels taken at the lowest and highest points of each section of the city will generally be sufficient.

The problem of fixing upon the source of supply generally involves the comparative consideration of all possible sources. The water of streams or lakes should be submitted to a chemist and bacteriologist for examination, as should samples of water from test wells if there is a probability of using ground water. The banks of streams and lakes and of their tributaries should be examined for sewers, drains, outhouses, slaughter-houses, or other sources of pollution. If a surface supply is in question, selecting the best location for the reservoir will require a survey of the limiting line of the catchment area; a line of levels along the valley, giving elevations relative to the city; test borings at possible dam sites to determine the character of foundation obtainable; and an accurate topographical survey of the dam site selected and of the reservoir location above it, that for the dam generally locating contours at 1-foot to 5-foot intervals, while 5-foot to 25-foot contours may be sufficient for the reservoir site. The location of all swamps, ponds, stables or cow-houses, buildings, roads, and other natural and artificial features should be determined. It will be necessary also to learn who are the owners of the properties in question, and in fact of all that are in the catchment basin.

If ground water is considered, the geology of the surrounding country should be carefully studied and the location and history of all wells in the neighborhood—their depth, character of strata which they pierce, and quantity and quality of flow from year to year. The probability of contaminated water entering the water-bearing stratum should be investigated, and special attention paid to this as well as to the mineral content and the chemical and bacterial analyses of the water.

If the supply be from an intercepting reservoir, the best conduit line between this and the city should be determined by survey. If an open conduit be employed, the survey will assume much the character of a railroad survey for a road with its grade and terminals fixed.

The location of pumping station, standpipe, distributing reservoir, and some other features of the system will call for special investigations and surveys, the latter mainly consisting of measurements of the distances and relative elevations between the proposed sites and the city.

Estimation of the quantity of water flowing in rivers is made by obtaining, with either floats or current meter, the velocity in a cross-section whose area has been measured. River gagings should be made during the lowest water if possible, and compared with calculations of the probable minimum yield to be expected from the drainage area. Gagings of certain rivers extending over long periods of time can be obtained from government reports or the records of water-power companies; but such rivers are unfortunately few.

All obtainable rainfall data should be collected; and one or more rain gages should be established in the catchment areas under consideration as soon as the investigation is begun and maintained throughout the time of the investigation, designing, and construction; and it will be desirable to continue their use on the watershed adopted, after the plant is in service.

The yield of a small stream may be obtained by the use of a weir; that of a well, by catching the flow in casks of known capacity, and noting the number of times they are filled in a given period, or by receiving the flow in a flume and measuring it by a weir. The natural flow of wells is generally increased, both during the test and for practical use, by a pump, either surface, deep-well, or air-lift.

The testing of a well should extend over at least a week, with several gagings daily, if an estimate at all accurate is desired; and tests are sometimes continued for a month or more, during which time the level of the water in the well should remain constant while the rate of pumping is uniform.

Art. 74. Selecting the Source of Supply

If the city is upon or near a river or lake, this forms the most obvious source of supply; but unfortunately there are few

in the settled parts of the country which are not more or less polluted, and there is a probability that settlements will before long be established along rivers in sections not now populated. Many cities which originally drew their supply from their own river-fronts have been forced to abandon this supply on account of the fatal epidemics of disease clearly traceable to it; and many others are eagerly searching for purer supplies for the same reason. A clearer foresight and shrewder economy would in many cases have led to the acquiring of drainage basins and reservoir sites when they could have been obtained at a much lower cost than must now be paid for them. Whatever the opinions of the citizens, it would seem to be the engineer's duty to place the river as the last rather than the first alternative source, and plainly and forcibly to state to the public his reasons therefor. If river water is to be used, purification, either immediate or in the future, should be considered essential.

The present quality of a river supply may be learned approximately by chemical and bacterial analyses, in connection with a searching investigation for all causes of contamination. It is desirable to obtain analyses not only during ordinary periods of flow, but during low and high water also. In determining the quantity of flow in a river it must be remembered that low water generally lasts for several days or weeks, while the maximum rate generally continues but a few hours. If no storage is to be supplied, particular care must be taken to learn the very lowest rate of flow attained by the river, and a margin of safety should be used in connection with this.

In testing the quality of a ground water, analyses should be taken during the quantity test; and particularly at the end of this, since the quality often changes as the flow continues, owing to differences in the composition of the near and distant parts of the water-yielding stratum.

Before or while making exhaustive tests of the quality and quantity of water available from different sources, an estimate of that required should be prepared. The estimate of population can be approximated as suggested in Art. 7; a considerable extra allowance for growth being made for small cities. The

decision as to quantity of consumption per capita is a most difficult one. If meters are to be used, and every effort made to keep the consumption within reasonable limits, 60 gallons for residential and 80 to 100 for factory towns should be sufficient; but many engineers assume for large cities a rate gradually increasing to a maximum of 150. If no efforts at limiting consumption are made, the consumption may reach any figure short of a thousand gallons per capita, and the only logical conclusion would be to furnish all the water available.

In case a water supply of good quality can be obtained in sufficient quantity for domestic consumption only, but water of a more polluted supply from a river is available also, the latter may be used for an auxiliary supply for factories, street and sewer cleaning, public fountains, and similar purposes; although great care should be taken that it be excluded from all faucets or other contrivances by which it might be obtained for drinking-water by careless citizens. Such a supply, for fires only, has been introduced at a number of cities; and for manufacturing purposes at New London, Conn. Salt water has been used for street-sprinkling and sewer-flushing in a number of English cities, where it is said to prove very satisfactory, 1 gallon of sea water laying the dust as effectively as three or four of fresh. The chief reason for introducing an auxiliary fire system is not, however, scarcity of supply (in fact, many such systems draw their supply from the regular mains), but the possibility of thus obtaining greater pressure and rate of discharge; and the use of such a system in no way affects the selection of a source of supply for domestic consumption.

In searching for a watershed to supply surface water, it is desirable to find one from which the water can be led by gravity, and this will generally be at the head waters of a stream passing through or near the city, or of one of its tributaries. This should be comparatively free from occupants, and preferably wooded and without swamps. The drainage area necessary may be calculated by dividing the maximum annual supply desired plus evaporation from the reservoir by the expected

yield per square mile. This yield may be the average annual yield, but not unless a very large storage reservoir is provided. It would be better to use the minimum average yield of three or four years. Great care and judgment should be used in estimating the rate of yield, all rainfall, evaporation, and stream-flow data available being used as outlined in Art. 35. The most reliable data are those obtained by careful run-off or stream flow gagings on watersheds in the same section of country and general topographical location, correcting these for different areas of water surface if necessary. If no such data have been obtained, use may be made of the general rainfall and run-off records for that section of the country, as obtained from government records. The less reliable and specific the data, the greater the amount of surplus catchment area which should be provided above that estimated.

The size of catchment area having been determined, a point should be looked for in the drainage valley which offers a favorable location for a dam and reservoir and above which the extent of drainage area is at least that found necessary. This point should also be at such elevation above the city that water can be delivered there by gravity and with the desired pressure. If one area cannot be found sufficiently large to furnish the entire supply, several may be used, the nearer they are to each other and to the point of utilization the better. A large city may be compelled to go a long distance for the necessary supply; as did New York City and Los Angeles, whose supplies are impounded 92 and 226 miles distant, respectively.

The best reservoir location is one at which the desired quantity can be stored on the least surface area, and where the shores of the reservoir will be fairly steep; also where the impounding dam may be short, have a solid foundation, and be constructed largely of material found near at hand. A bowl-shaped basin among the hills or mountains, having a narrow gorge for its outlet, best fulfills these conditions. Reservoirs have been built in a rolling country, however, where the length of the dam is as great as that of the reservoir, and the

maximum depth but a few feet; but such reservoirs lose much of their stored water through evaporation and seepage, and it is liable to acquire undesirable qualities.

The geological conditions of a reservoir site should be such that there is little danger of leakage; the strata being synclinal rather than anticlinal, and containing no fissures or open faults.

In searching for a ground-water supply, if shallow wells or filter galleries are proposed, no data concerning the flow to be expected can be obtained from points more than a very few miles away, except as these have thrown light on the general laws of ground flow. Test wells and pumping and a thorough knowledge of the soil conditions and the source of the water reaching the place in question should form the chief basis of estimating the flow. In this, as in tests of deep wells, pumping should be continued for a considerable time; and it is desirable to sink another observation well at some distance—say 1000 feet—away, and note the fluctuations of ground-water level in it. If this constantly falls while pumping at a uniform rate is continuing, it indicates that not only is all of the regular ground flow being pumped, but the ground storage is being drawn upon. The water surface in the observation well may be lowered during the first day or two of pumping, even if this be less than the rate of ground flow, but after the conditions of flow to the well become established it should remain stationary. If the pumping could be continued at the maximum rate which it is possible to maintain without drawing upon the storage, this would equal the ground flow at that time. But it must be remembered that shallow wells are much more affected by droughts than are deep ones.

Of deep wells also the only sure knowledge can be obtained by actual test; but if there are others in the vicinity, much may be judged from their performance. If these tap water-bearing sandstones or other rock strata, the probabilities are that wells of similar capacity and characteristics may be driven to the same rock over a considerable section of country; as in the case of the Dakota, St. Paul, and other sandstones in the North-central states. If the wells are in glacial deposits, the extent of the strata

which they tap is very uncertain and can be known only by actual investigation.

If the amount of water desired is small, the possibility of obtaining it from springs should not be overlooked, and suitable ones should be searched for. On the other hand, the area that need be investigated for supplies is generally limited by the small amount of money available for constructing conduits in a small plant.

It will generally be desirable to make comparative estimates of the cost of using each of the available sources; never forgetting that quality should outweigh any financial considerations where the water is for domestic use.

Art. 75. The General Design

The location of the source of supply will generally determine whether the system will be a gravity or a pumping one; and the two may be combined, either when there are two sources of supply, or when the territory to be supplied is at different elevations. In the latter case there will generally be a high-service distributing reservoir into which the water is pumped; in the former the pumping and gravity supplies may be distributed by practically two separate systems, or, as is more often the case, pumping is resorted to to supplement the gravity supply when this becomes deficient. If the storage reservoir is at a great distance from or elevation above the city, a smaller distributing reservoir is desirable. The top of a hill or ridge immediately above the city and 150 to 300 feet higher forms the best location for such a reservoir; or a flat slope on a side hill may be used. If such location is not obtainable and the storage reservoir is more than 300 to 350 feet higher than the city, in which case a distributing reservoir is desirable to prevent a pressure head dangerous to ordinary plumbing, this may sometimes be located lower down the valley of the impounded stream and formed by a small dam across this valley, the old stream-bed serving as a conduit to connect the two reservoirs. But if the watershed

above the distributing reservoir is liable to pollute the supply, the distributing reservoir should be placed out of the channel of flow.

If pumping is employed, a reservoir or standpipe is extremely desirable, and should be omitted only when it is impossible to obtain the money to pay for it; and even then its future construction should be provided for, and carried out as soon as possible. Whether reservoir or standpipe is provided will generally depend upon the local topography. A reservoir is decidedly to be preferred if a hill sufficiently high for one is near the city; but if there is no such hill, a standpipe will be necessary to give the desired elevation and head of water. A standpipe may be placed anywhere in a city, as it occupies but little ground space. It is more efficient the nearer it is to the center of the district most urgently requiring fire-protection—generally the business or manufacturing district.

The elevation of the reservoir or standpipe will determine the volume and range of fire streams in the city, since the pressure at the nozzle, plus the friction loss in the hose and pipes between this and the reservoir or standpipe, must equal the difference in level between the nozzle and the surface of the stored water. Knowing the volume and range of fire streams and the number of simultaneous streams desired, and the size and length of pipes between the nozzle and supply, the velocity in these last can be calculated and from this the friction head; and friction head plus nozzle pressure head plus elevation of the principal fire-risk district will give the desired elevation of the water in reservoir or standpipe. A standpipe should be carried at least 15 to 20 feet above this elevation, since the water level falls rapidly during fires.

In the case of a large city in a level country such a condition is practically impossible unless a number of standpipes be scattered throughout the city. (In such a location the supply will in every case, probably, be by pumping.) It then becomes necessary to obtain additional head by pumping during fires. This may be effected either by the use of steam fire engines, in which case the head in the water mains must be sufficient to

supply water to the fire hydrant at the maximum rate at which the fire engines can use it; or the pressure and volume given by the water-works engines may be increased when a fire breaks out, the reservoir or standpipe (if any) being at the same time shut off from the system. The objections to the latter plan are, that a longer time is generally consumed in raising the pressure at the pumping station than in starting a fire engine; and pipes and fixtures which showed no sign of weakness during ordinary service pressure are apt to be broken by the excessive fire pressure. Both these objections are found to be seriously applicable to four out of five of the smaller cities and towns.

If the supply for a pumping system be from ground water or other source not subject to occasional pollution with considerable suspended matter, or if it be filtered, an excellent arrangement is to place the pump and the reservoir or standpipe on opposite sides of the city, and use the direct-indirect system. If the main break near either pump or reservoir, the other of the two can then supply the entire city; and with a proper arrangement of piping it will be impossible for any one break to deprive more than one city block of its supply. Also a more generally uniform fire pressure can be obtained thus than if pump and reservoir were on the same side of the distribution system.

If the supply is from a river, however, and is not filtered the pumps should preferably discharge directly into reservoirs, that the water may have some opportunity for clarification when muddy before being delivered for consumption. At least two reservoirs, or one double reservoir, should be provided, and these should always be kept full while the river water is clear, to furnish the supply while it is muddiest; being if possible of such size as to furnish the supply until the river water again becomes clear. The further a river valley extends above the intake the longer will this period of turbidity continue. The period will vary widely with the character of soil and slope of the watershed, but for average conditions the turbidity will probably continue, after a heavy rain ceases, one day for each 30 to 50 miles of river above.

Where there is no location for distributing reservoirs in a river pumping system, it is advisable to place settling reservoirs near the river at the pumping station, and pump into these, and from them to a standpipe by the direct-indirect system. The same pumps could be used alternately for pumping from river to reservoir and from reservoir to standpipe; but greater economy could generally be obtained by using for the former centrifugal pumps of high efficiency, and high-duty pumps for the service pumping, both being run by the same boiler plant. A by-pass should be provided around the distributing and settling reservoirs from the pumping to the distribution main, to be used for direct pumping if it is desired to clean or repair the reservoirs, or to increase the pressure during fires; but if the water is unsafe unless purified, the purification plant should not be by-passed under any consideration.

An uncontaminated supply is more satisfactory and reliable than a purified one, but the latter is infinitely preferable to a contaminated one unpurified. Uncontaminated supplies are becoming very scarce, especially in the thickly settled parts of the country; and nearly all surface supplies should be purified. Purification works for a gravity supply may be immediately below a storage reservoir, but would generally be more accessible just above a distributing one. In the case of a pumping supply the purification plant is generally near the river and the pumps, secondary pumps raising the water to it from the river if necessary; although in some cases the plant is at or near the distributing reservoir into which the water is pumped, as at Louisville, Ky. If pressure filters be used, these may be placed near the pumps on the pumping main, water being pumped through them directly from the river. But the life and efficiency of both pumps and filters will generally be increased by first clarifying the water in a sedimentation basin.

Art. 76. Gravity Head Works

The investigation of a watershed calls for accurate data concerning its area and form. These may sometimes be obtained from state or federal publications; but in most cases

it is necessary to make the required surveys. The most important of these is a careful running of a transit-line around the boundary of the catchment area, and a line of levels (referred to the city datum) and transit line up the bottom of the drainage valley. From a map prepared from these data the catchment area above any proposed dam site can be scaled off, and its total yield estimated; and the head available at the city due to the elevation of the dam may be learned.

At the dam site selected, borings and test pits should be made to determine the character of the underlying strata. These should be located at several points over the site to be covered by the dam, and also over the reservoir area. An impervious stratum should not be pierced by the test holes; but if found to exist under the dam and reservoir site, other test holes a short distance below the dam site should be carried through or several feet into the impervious stratum to learn if its thickness be sufficient. If this be more than one-third the height of the dam and the material be solid, it can probably be relied upon as a foundation. If less than this be found, with a soft or porous material underlying it, further investigation should be made before finally selecting the site; although an earth dam would probably be safe upon a considerably thinner rock foundation if free from seams or cracks. The danger from a thin impervious stratum is twofold: its strength may not be sufficient to support the dam, or the head of water created may cause a leakage through it.

A thorough investigation should be made, if necessary, to render certain and exact the information concerning the geological formation at the dam site. Such conditions as are shown in Fig. 86 are inadmissible. In *a* leakage would almost surely occur under the dam. In *b* it would penetrate the rock and the under stratum in consequence of the head created behind the dam, and either a leak result or the under stratum become saturated and soft; in either of which cases the rock would be likely to yield under the weight of the dam. In *b* the dam should be carried down to a thick stratum of unseamed rock or moved to another location, as below the point *c*. In *a* the

core wall may be carried to the lower rock stratum, unless this also is underlaid by a porous stratum which outcrops within the reservoir area.

Not only the bottom, but the sides also of the valley should be carefully examined for seams, porous strata, or other conditions which would cause leakage under or around the dam.

From the size of the catchment area and the estimated yield, the amount of storage required can be estimated. This capacity is then used to learn the height of dam necessary. A carefully prepared contour map of the reservoir site is made, the contour interval being from 1 to 5 feet, depending upon the steepness of the ground slope. The dam is approximately located on this.

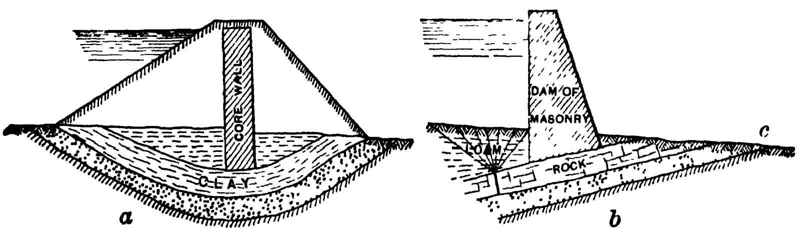


FIG. 86.—Unsuitable Dam Sites.

Beginning at the lowest contour, the area included by this above the dam is measured by a planimeter or otherwise, and the same is done with each successive contour; the volumes embraced between successive contour planes are calculated, and when the sum of all these volumes below a certain plane equals the amount of storage desired, this contour plane will be the elevation of the reservoir spillway. The water will rise such a distance higher than this as is necessary to discharge the maximum floods over the spillway.

If a storage reservoir of the desired capacity would require a very high dam, it may in some cases be cheaper to construct two reservoirs, either one below the other in the same valley, or in different valleys.

If the back-water from the reservoir will cause shallow ponding of water along the banks of the tributary stream,

the channel of this should be walled in and the banks raised above the water level of the reservoir; and the same treatment, or excavation and paving, often combined with filling as at *b*, Fig. 87, should be applied to all shallow water within the limits of the reservoir. If a shallow pond is formed, as at *a*, it will generally be better to fill this entirely, as shown by the dotted lines. Sufficient material for this will in most cases be obtained in clearing off the reservoir site and grading the banks. The last few feet nearest the slope of this fill, however, should be of gravel or earth free from organic matter. Embankment slopes should be paved, with dry slope wall or otherwise; and it is desirable to treat the entire shore of the

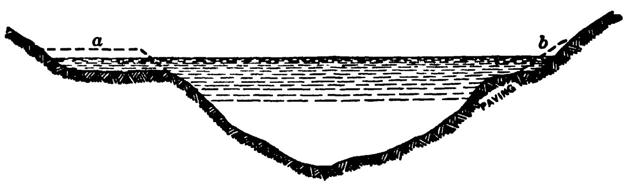


FIG. 87.—Reservoir Cross-section.

reservoir in this manner, for 2 feet above and a few feet below ordinary water limit.

The decision as to the material of which a dam should be constructed will, to a large extent, depend upon the size of the dam, character of the foundation, the material at hand, the accessibility, and the money available. If bed-rock is near the surface and can be quarried for construction near by, a masonry dam is easily possible. If hardpan or clay is near the surface and good embankment materials near at hand, an earth dam is practicable. If rock exists for a foundation and for masonry, and also soil adapted to embankments, either construction is possible. Where labor, cement, and sand are reasonably cheap an earthen embankment more than 70 or 80 feet high will probably cost more than a masonry dam, owing to the great width of the base and volume of material in the former. If the dam is short and the spillway must be placed in it, this, which must generally be of masonry, may

form so large a proportion of the dam that constructing the whole of masonry as a weir dam would cost but little more and would be much safer.

If no rock, hardpan, or clay exist within 40 or 50 feet of the surface to form a foundation and impervious bottom for the reservoir, it will probably be well to endeavor to find another

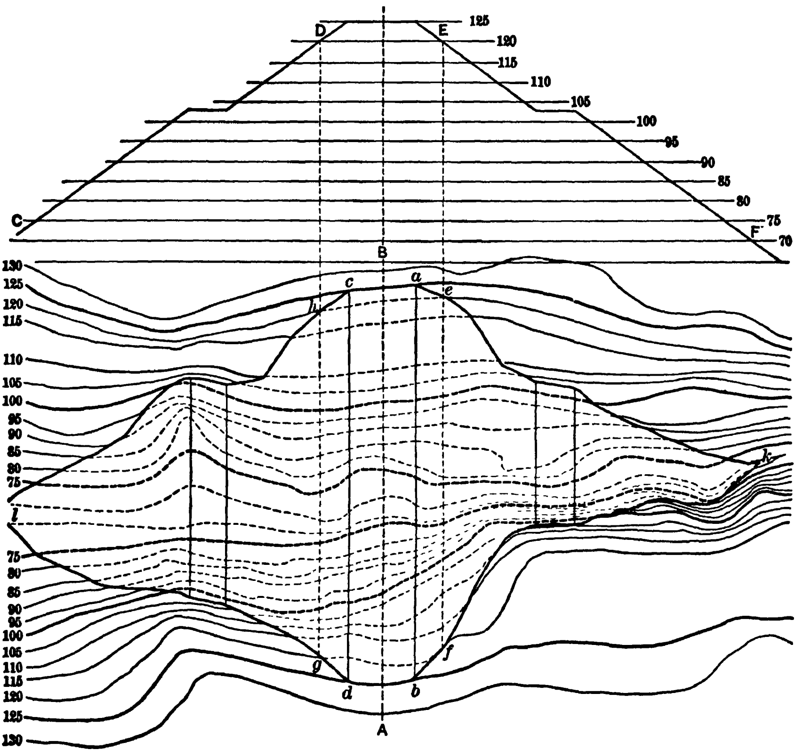


FIG. 88.—Estimating Contents of Earth Dam.

site. If the site selected be not underlaid with such a stratum of considerable thickness it would be useless to place there a tight masonry or earth dam of considerable height; but masonry 10 or 15 feet high may be constructed on a timber foundation protected by sheet piling, although this construction is to be avoided if possible. In such a location a rock-fill or timber dam may be used to raise the water level if no storage be required.

The general form and cross-section of dams has been already considered in Chapter XIII. The cubic contents of a given dam can be most readily ascertained by the use of a contour map of the site, to a large scale. The method is illustrated in Fig. 88. On the prolonged axis, *AB*, of the dam a cross-section of the same, *CDEF*, is drawn to the same scale as the contour map and with *AB* as its vertical axis. If the crest of the dam is to be at an elevation, say, of 125 feet above datum, horizontal lines are drawn at intervals of 5 feet (if this be the contour interval) beneath this and numbered with their proper elevations. The points of intersection of the 120 line are projected down to the map, and where the projection lines cut the 120-foot contours, as at *e*, *f*, *g*, and *h*, are points in the junction of the dam and the earth surface. *ef* and *gh* are connected; *a*, *b*, *d*, and *c*, are found and connected in the same way, and *a* is connected with *e*, *b* with *f*, etc., the outline *aekfbdglhc*, being thus formed. The areas *acdb*, *ehgf*, etc., are now measured and treated as parallel sections of a prismoid in calculating the cubic contents of the dam. The contours *ac*, *bd*, *fg*, and *eh* are taken as the ends of the horizontal sections. Instead of using surface contours, those of the ground as cleared or excavated, or of the rock, if it be a masonry dam, should be used; or the surface contours may be used, and to the resulting calculation may be added the amount to be excavated and then refilled as embankment or masonry.

The upper face of earth dams should be protected from wash, weeds, and ice by paving. This is generally in the form of a dry wall 12 to 24 inches thick, composed of a single layer of flat stone, carefully laid by hand on a bed of coarse gravel or broken stone 6 to 20 inches thick. A bed of concrete 4 to 12 inches thick is better than the broken stone; and when this is used brick is sometimes substituted for the dry stone wall. A water-tight lining is desirable if there is no core wall and the embankment is not sufficiently impervious.

The outer slope of a dam is generally sodded to prevent wash by rains; although in some cases this also has been paved. If there is a berm, a paved gutter should be placed along the

inner edge of this, leading to a drain which discharges upon the natural surface below the dam.

The face of a masonry dam may be vertical, sloping, curved, or stepped. The first is applicable to very low dams only, the second to dams up to 20 to 30 feet in height if never overflowed, but only 10 or 15 feet if ever acting as weirs. Weir-dams from 10 or 15 to 20 or 30 feet high may be stepped in front or given an ogee form. Dams more than 30 or 40 feet high should be given forms calculated for greatest economy; as the "economical" or the new Croton profile in Fig. 71; and if to act as weir dams, should have an ogee or other curved face, similar to the Austin or Holyoke dam. The face stones may all rest on horizontal beds, but in weir dams, at least, it is better to make the joints radial.

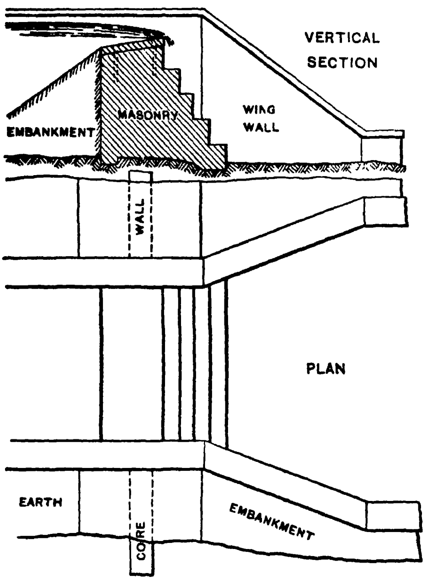


FIG. 89.—Spillway in Earth Embankment.

The crest- or cap-stones should be heavy and substantial, and where there is great danger from heavy logs and ice they may be bound together, as in the Holyoke dam. For spillways in storage reservoirs, however, the construction shown in Fig. 89 will generally be sufficient, the steps being sometimes replaced by an ogee face for high spillways. If the main dam be of earth and a masonry core wall be used, it should be joined to the masonry of the spillway; or if there be no core wall, a spur wall should be carried for some distance into the center of the embankment from each end of the spillway. A spillway over a natural bed rock at one end of the dam is generally preferable to a weir.

The cut-off flanges for pipe passing through an embankment or dam may be made as shown in Fig. 90. A thin sheet of lead or other yielding and durable material should be placed between the casting and the pipe, and between the flanges of the casting, so that no water may work its way between the pipe and cut-off flange.

But one pipe, an outlet main, may be used in small reservoirs, a blow-off branch being placed on this just outside the reservoir and controlled by proper valves. The inner end of such pipe must be at the lowest point in the reservoir and flush with the bottom, in order that the water may all be drawn off. It

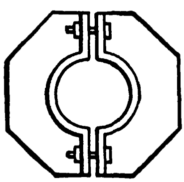


FIG. 90.—Flange for Pipe in Embankment.

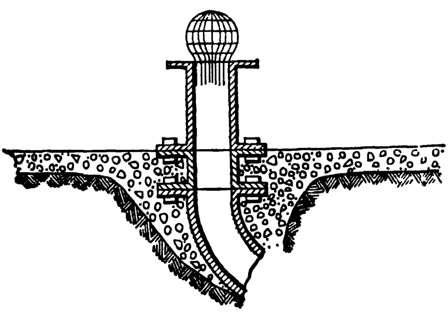


FIG. 91.—Outlet Pipe for Small Reservoirs.

should, however, be supplied with a removable extension rising at least a foot or two above the bottom, to exclude mud and other deposits from the outlet main during service. This riser should terminate in a screen to keep out fish and other large matters. The gates controlling the flow from the reservoir can be placed outside the embankment in a manhole or gatehouse. This is the simplest arrangement practicable, and is applicable to the smallest reservoirs only; and even in these a separate "mud" or "waste" pipe is preferable.

Such a screen as is here referred to cannot well be cleaned without removing it entirely. Also if any break or leak occur in the outlet pipe above the gate, the flow through it cannot be stopped except by emptying the reservoir, before which the bank would probably be destroyed. It is hence better to provide gates on the outlet pipe *inside* of the reservoir; and duplicate

screens which can be removed for cleaning and replaced without leaving the outlet unprotected; and this is always done in large reservoirs. A vertical flat screen does not become obstructed so quickly as a horizontal one and is more easily removed and cleaned and is consequently preferable. Such a screen may be made of, say, No. 10 copper wire, ¼-inch mesh, in a steel

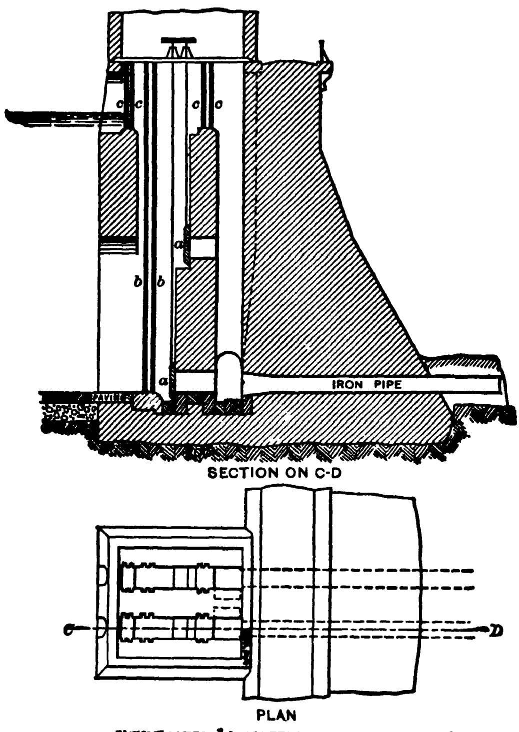


FIG. 92.—Gatehouse, East Branch Reservoir, New York City.

frame; but a copper plate, punched to form a screen, cleans more easily. Wire screens in frames of almost any size are carried in stock by companies dealing in valves and other water-works supplies.

The form of gatehouse employed is ordinarily on the general plan of that shown in Fig. 92. Any one screen can here be removed, a duplicate one meantime being in service.

The discharge from the reservoir can be passed through any or all outlet pipes, each of which is controlled by a gate worked from above by a stem and wheel. Water enters the gatehouse from the reservoir through several openings, *aa*, in the wall, which are placed at different elevations and controlled by sluice gates to permit of drawing water from any depth, and thus securing the purest and coolest. The lowest one should be a little lower than any part of the reservoir bottom, and is used for emptying the reservoir, or drawing off the foulest water before the "turn-over," through a mud or waste pipe, which discharges below the dam, generally in the old creek-bed.

The outlet pipes and gatehouse demand the most careful attention, the firmest foundation, and the most substantial construction. The foundation of the gatehouse should rest on the same material as does the dam or core wall. The bottom should be absolutely water-tight; if not on rock, a broad thick bed of concrete forms probably the most desirable foundation. The wall of the gatehouse should be of water-tight concrete or of small uncoursed ashlar, in the best of Portland cement-mortar, with walls of such thickness that no stone reaches entirely through them; the idea being to make the walls water-tight and sufficiently strong to support the outside water pressure when the gatehouse is empty, or the pressure of ice. The construction of the gatehouse should be such as to prevent its injury by ice if the water in the reservoir rises or falls while frozen over, or by the expansion of ice between it and the dam. To insure this it is generally better, if the dam be of masonry, to have the gatehouse built against or as a part of it. In the case of earth dams, the gatehouse is generally at the inside foot of the embankment, and communication with it is afforded by means of a foot-bridge from, and on a level with the top of, the embankment. In a few cases the gatehouse has been formed by sinking a well in the rock at one end of the dam, and filling it through a tunnel or open cut carried from it into the reservoir; this plan avoiding the weakening of the dam by the outlet pipes. In the Oak Ridge (East Jersey Water Company) reservoir this plan was adopted, the gatehouse

and inlet and discharge channels being more than 40 feet deep in solid bed-rock. The gatehouse is floored over, generally on a level with the top of the dam; and is covered with a small building to protect the gates and exclude meddlesome intruders. This building is frequently given an attractive appearance.

The outlet pipes should make a water-tight connection with the gatehouse, and be placed on a perfectly firm foundation, that they may not be broken by settlement. The bottom of the reservoir, for several feet in front of the gatehouse inlet opening, should be paved to prevent washing.

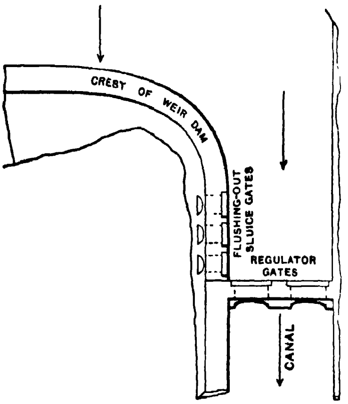


FIG. 93.–Headgates and Flushing-out Sluices.

Where a storage reservoir is not necessary and a diverting dam only is provided, the water is generally drawn off from at or near the surface of the stream. For this purpose, instead of a gatehouse and pipes, an opening is ordinarily made in one end of the dam, leading to an open conduit, this opening being controlled by sluice gates. In addition to the gates it is generally desirable to place in front of them an upright grooved frame with flash-boards sliding in the same, and a contrivance by which an opening can be left between flash-boards at any elevation at which it is desired to draw off the water. This opening should generally be kept a foot or two below the water surface, that both floating brush, leaves, etc., and the heavier matter carried in suspension may be excluded from the conduit. As sand and gravel will probably collect in front of the flashboard weir, another opening is generally provided in the bottom of the dam near this weir for flushing out the deposit at intervals.

ART. 77. PUMPING STATION AND INLET DETAILS

A pumping station should generally be located as near as possible to the body of water from which it draws, to reduce both friction in the suction pipe and the possibility of its leaking air and reducing the vacuum. Boilers should be above the reach of floods; and their foundation should be extremely firm, on account of the great weight to be supported, which may amount to 250,000 pounds or more for each boiler. The pumps also must be protected from water, but it may be necessary for them to be below high water (see Art. 63), in which case they are generally placed in a water-tight pump pit. The walls of this may be of concrete, or stone masonry, or of sheet iron or steel; and the well is usually circular in plan, this giving the greatest strength. To reduce the size and cost of the well, the pumping engines only are placed in it, and these are generally vertical or centrifugal engines. The stairway to the bottom of the well can be made to follow the wall in a large spiral. Water-tight joints must of course be made where the suction pipes pierce the wall of this well.

If the pump is on the surface, the suction pipe should be carried several feet beneath the ground level before leaving the building, and be kept at this depth until reaching the water, to prevent its freezing in winter. The suction pipe should be larger than the discharge; and if of considerable length should be of such size that, when all the pumps are in service, the velocity of flow through it will not exceed 2 to 4 feet per second for 12- to 36-inch suctions respectively. It should be laid with unusual care to obtain tight joints. It should not terminate in a river or lake wall nor near the bank or shore, as the water here is more liable to contain fresh sewage and floating matter than in mid-stream. The best location and design of the inlet will vary with the circumstances. It should be placed where the water is purest; in a lake especially, this will generally be the furthest possible from the shore. It should be in deep water, as this is ordinarily cooler; but should not be near the bottom, as the most sediment is carried there. An excellent plan is

TABLE 43

SPACE OCCUPIED BY PUMPS OF DIFFERENT TYPES

Centrifugal Pumps.

Size (Discharge Pipe.)	Gallons per Minute Capacity.	Millions of Gallons per Day.	Floor Space, Inches.	Height, Inches.	Shipping weight, Pounds.	List Price.
		Horizontal Pumps, including Pulley.				
5	625– 750	0.9– 1.08	34×54	35	940	$ 165
6	900– 1080	1.3– 1.56	37×55	40	1180	200
8	1600– 2000	2.3– 2.9	45×65	46	2065	310
12	3600– 4300	5.2– 6.2	63×71	52	3615	500
18	8800–10590	12.7–15.3	90×105	64	9000	1300
		Vertical Pumps.				
5	735	1.06	35×45	50	785	216
5	1050	1.51	37×55	55	1100	285
8	2000	2.88	45×60	65	1710	445
12	4200	6 05	63×75	72	3200	700
18	10000	14 40	98×125	84	7000	1585

Horizontal Cross-compound Engines (Allis-Chalmers).

Assumed head, 200 ft.; piston speed, 250 ft. per minute; steam pressure, 140 pounds.

Capacity, Million Gallons per Day.	Diameter of Low-pressure Cylinder, Inches.	Diameter of High-pressure Cylinder, Inches.	Floor Space. Width.	Floor Space. Length.	Capacity, Million Gallons per Day.	Diameter of Low-pressure Cylinder, Inches.	Diameter of High-pressure Cylinder, Inches.	Floor Space. Width.	Floor Space. Length.
2	22	10	11′ 7″	23′	7	42	20	15′ 0″	32′
3	28	13	12′ 9″	25′	8	44	21	15′ 6″	34′
4	32	15	13′ 6″	27′	9	47	22	16′ 3″	36′
5	36	17	14′ 0″	28′	10	50	23	16′ 3″	36′
6	38	18	14′ 5″	30′					

Vertical Triple Expansion Engines (Allis-Chalmers).

Direct-flow type. The dimensions of the "pier" and "deep pit" types are somewhat different.

Capacity, Million Gallons per Day.	Stroke, Inches.	Head, Feet.	Minimum Distance from Basement to Engine Room Floor.	Height above Engine Room Floor.	Approximate Floor Space.
6	36	300	12′ 0″	24′ 0″	26′×15′
8	36	165	12′ 0″	24′ 0″	27′×16′
10	48	300	14′ 0″	28′ 0″	32′×20′
12	48	334	14′ 0″	28′ 0″	33′×20′
15	48	163	14′ 0″	28′ 0″	33′×20′
20	48	195	16′ 0″	28′ 0″	33′×21′
20	60	218	17′ 0″	33′ 6″	36′×23′
25	60	188	17′ 0″	32′ 6″	36′×23′
30	60	81	17′ 0″	31′ 0″	36′×23′
30	66	173	18′ 0″	35′ 0″	36′×23′

that of terminating the inlet in a tower somewhat similar to a reservoir gatehouse, there being several openings at different levels through which the water can be taken as desired. In the Nashville, Tenn., waterworks this tower is hexagonal in plan, 10 feet interior diameter and 85 feet high, of stone masonry on solid rock. At Cincinnati is a masonry intake about 140 feet high, the range of river height being over 70 feet. The intake for the Chicago water works is 4 miles from shore, formed of a circular double steel shell 70 feet inside diameter, the space between the shells being filled with concrete so as to form a wall 24 feet thick; the whole resting in 40 feet of water, and being 50 feet high to the bottom of a masonry superstructure. The St. Louis masonry inlet tower is about 50 feet high, and is oval in plan, with an ice-breaker pointing up-stream. The Cleveland intake is of steel, somewhat similar to the Chicago one, 100 feet outside and 50 feet inside diameter, located in 49 feet of water.

Such inlet towers must be substantial and massive enough to resist water currents, ice, or floating logs, and are consequently expensive. They are also an obstruction to the current in a river, and to shipping in a navigable water in which situation they must be provided with a light-house.

Instead of a tower, a submerged crib is frequently used, especially in smaller plants. This should be placed where there is least danger of silting, and it is desirable to make its height about one-third the depth of the water, but not less than 3 nor more than 15 feet high. This structure is essentially a wooden crib, weighed with stone and anchored to the bottom in some way to prevent movement by tides or currents, this being effected by surrounding it with coarse riprap, or piles, or bolting it to bed-rock. It may have openings on the sides; but it is in most cases preferable to make these tight and take water through the top only, which is provided with a coarse grating. The suction or intake pipe rises into the center of this crib.

The total area of the inlet openings in the crib should be considerable to prevent the formation of a vortex in the water

above, which will cause floating matter to be sucked into the main; and to prevent the entrance of "needle" or "anchor" ice, which closes the openings in the pier or crib or in the intake pipe, or may reach and stop the screens or even the pump. A very little motion will serve to carry the ice needles down and into an intake crib, and several cities have suffered water famines for several hours and days on this account.

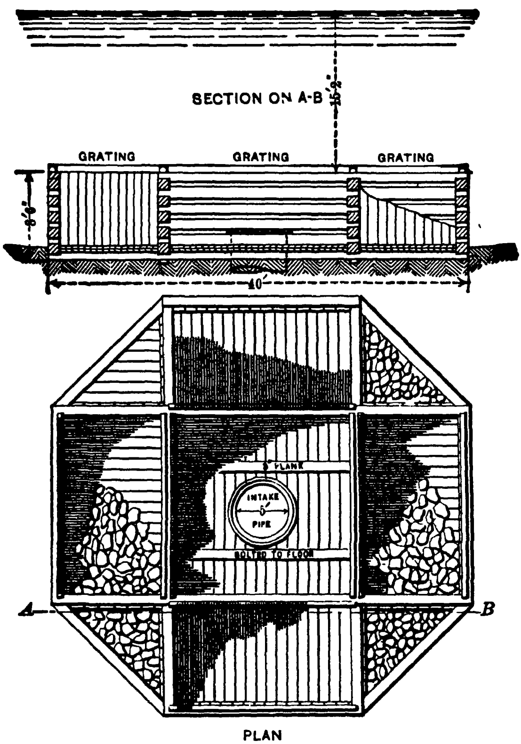


FIG. 94.—Erie Intake Crib.

Intakes for small plants in shallow water are frequently not provided with a crib, but are simply an upright extension of the suction pipe, brought up to a short distance below the water surface and surrounded with concrete or riprap, and in some cases with piles to protect it from boats or floating logs. A great number of small cities use such intakes, and they give fairly good service in many cases. In fact, if the water is

quite shallow the use of a crib may be practically impossible in Northern streams, where the going out of the ice would carry the crib with it. If the expense is not prohibitive it is, in such a case, desirable to place a low weir dam just below the intake, to raise the water surface, permit the use of a crib, and prevent damage to the intake from ice and floating timber.

To prevent the stoppage of an intake pipe by ice, several devices have been used. In one, compressed air is carried by a small pipe, which is laid in or fastened to the outside of the intake pipe, and delivered under the horizontal screen which covers the intake opening, the rising bubbles preventing the ice from collecting. In another, steam from the engine room is applied in the same way.

An intake pipe or tunnel leads from the intake to a suction well, from which the water is pumped; or the pipe may act as a suction pipe and lead direct to the pump, this being the common arrangement for small plants. Large lake cities also take their supply through tunnels several thousand feet long; but smaller supplies are generally drawn through pipes, either cast or wrought iron, laid in the bed of the stream or lake. These should be laid in trenches, to protect them from currents or shifting sands, and are sometimes covered with rip-rap as a further protection.

If filter cribs are used these take the place of intake cribs. The intake pipe is similar to that from an ordinary crib.

Art. 78. Ground-water Plants

Except where artesian wells flow under considerable head, it is generally desirable to be able to draw down the ground water as low as possible in the well, thus increasing the flow. Since pumps can raise water by suction only 20 to 25 feet with any efficiency, it is frequently necessary to lower the pump to meet these conditions. A surface pump may be used, placed in a pump pit, unless it is desired to draw the water down to a distance of more than 40 or 50 feet below the surface, requiring a pit more than 20 or 25 feet deep, when such construction is

not often employed, but deep-well pumps are lowered into the wells or the air lift is used. (An exception to this is the Rockford, Ill., plant where centrifugal pumps were placed in a pump pit 80 feet deep.) The efficiency of deep-well pumps and air lift is so much lower than that of a good reciprocating pump, and the latter is so much more accessible for repairs, that its use is recommended wherever possible. Where a large dug well is the means of supply, the pump may be placed in this, the boiler being on the surface, and the whole roofed over. This is likely to cause a pollution of the water, however, and a better plan in most cases is to place the pumping station near the well, connecting the two by a suction pipe, and roofing over the well.

If the wells are tube-wells, they are ordinarily coupled to a main collecting pipe which passes by and near them, the short connecting branches being furnished with valve gates, that any well may be put into or out of service. Unless deep-well pumps are used, the collecting pipe and branches should all be perfectly air-tight—a result often difficult of attainment. In spite of all efforts, some air is likely to leak into the collecting pipe, and this air should be removed in some way; especially if the collecting pipe be connected directly to the pumps, as the presence of air in the pumps will cause them to "pound." The only practicable method of accomplishing this seems to be to place an air-drum in the pipe at its highest point, and remove the air from this with a vacuum pump.

If the pump be connected to the collecting pipe and draw directly from the wells, a sand-box or sand interceptor should be placed between the pump and the wells to prevent sand from entering and cutting the pump valves and plunger or piston. The pump may draw from a suction well which has been dug to somewhat below the depth to which it is desired to lower the ground water, the collecting pipe being laid at about the level of the pump and its vertical end carried down to near the bottom of the suction well. By this plan the water is siphoned into the suction well, which also acts as a small reservoir to permit of higher rates of pumping for short intervals than could

be obtained directly from the wells. The suction well also serves to intercept the sand. The collecting pipe is apt to fill with air at its highest point, and this should be near the pumps and provided with an air-drum, and arrangements made for removing the air at intervals.

The distance by pipe from the pump to any well should be as short as possible, and the collecting pipe of such size that the velocity of flow in it shall not be greater than 2 or 3 feet per second, that the friction loss between well and pump may be a minimum. The best arrangement for obtaining this result is to place the pump at mid length of the collecting pipe. If two or more water-bearing strata are tapped by the wells, these may be all connected to one collecting pipe; but if the water rises naturally much higher from one stratum than from another, it is better to provide a separate suction from each, and, if more than one pump is used, so arranged that either pump can draw wholly from either stratum or from both combined.

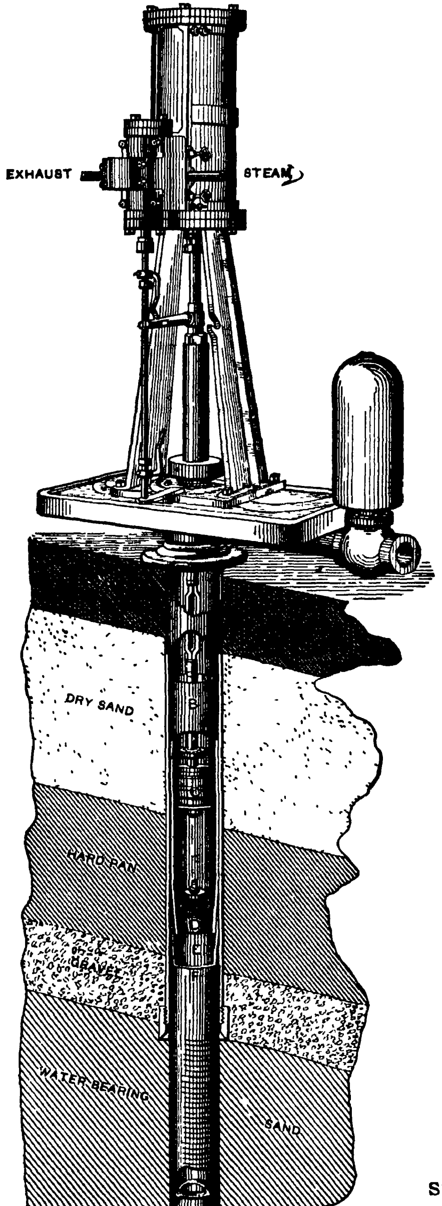
If deep-well pumps are used one must be placed in each well, and be driven by separate engines or working-heads. This will require a building of some kind over each well; and either an engine to each pump, or one engine transmitting its power by rope, belt, water or air pressure, or electricity to the various working-heads. For this reason deep non-artesian wells should be given large casings and powerful pumps, that their number may be reduced to the minimum.

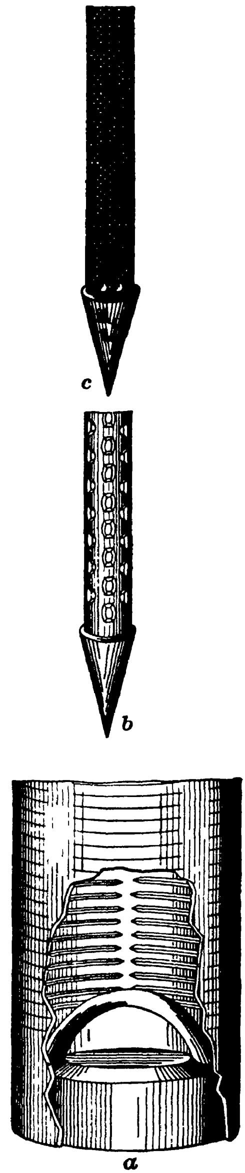
The location of wells has already been referred to. It is generally desirable to drive them in low ground, as the suction lift of the pumps is thus reduced at the least expense of pump-pit excavation. They should never be placed along the direction of ground-water flow, but as nearly as possible at right angles to it. The most desirable spacing will vary with the soil, volume of flow, and depth to which pumps lower the water in the well. It should generally be such that the ordinary pumping of one well will have little effect upon the simultaneous yield of its neighbors. This is particularly true of deep and expensive wells. The sinking of an additional well

between any two others will, however, almost always increase the total supply, although it may be but slightly; and if the wells are shallow and cheaply driven they may be placed quite close together. At the Spring Creek station of the Brooklyn water works, one hundred 2-inch wells, 30 to 42 feet deep, were placed in two rows 14 feet apart, the interval along each row also being 14 feet; and the same spacing was used at the Jameco station, where are 183 2-inch wells from 27 to 73 feet deep.

Water enters a well through a strainer, which contains openings in the form of circular holes or slots. These openings must be sufficiently small to exclude at least the coarser half of the soil material in the stratum from which water is drawn, or else, if the holes are larger, the strainer pipe is covered with a fine gauze screen of brass wire, which is protected by an outer heavy screen or a tube of thin pierced metal called a jacket. Better than the latter is a brass pipe provided with slots, wider on the inside than on the out to prevent sand clogging it. Instead of a gauze screen, perforated pipes wrapped with No. 14 wire, ten to fourteen turns to the inch, is a construction extensively used in the lower Mississippi valley.

The strainer is generally of a length equal to the thickness of the water-bearing stratum, if this is thin. The larger the area of strainer, the greater the amount of water that can enter with a given velocity and proportionate loss of head. On Long Island from 0.1 to 0.15 gallon per minute per square foot of strainer surface can be obtained for each foot of head by which the ground water surface is lowered at the well by suction. (This is called the "specific capacity.") If a million gallons per day per well is to be pumped, the well should be 14 or 16 inches diameter to keep the velocity in it to less than 1.5 per second. If it be 16-inch, the specific capacity 0.1, and the ground water be lowered 20 feet by pumping, the length of strainer must be $\frac{1,000,000}{24\times 60\times 0.1\times 20\times 3.1416\times 1.33}=83$ feet. If the specific capacity be 0.15 and the ground water be lowered 40 feet, the strainer could be shortened to 28 feet. Specific capacity varies with





STRAINERS
a. Cook Strainer.
b. Drive well point before gauze and jacket are placed.
c. Point and strainer.

FIG. 95.—Deep Well with Pump.

coarseness of grain of water-bearing stratum and the freedom with which the water enters the strainer.

The size of pipe, length of strainer and number and spacing of wells are seen to be inter-related to the amount of water desired or that can be secured from each well, the depth and porosity of the water-bearing stratum and the amount of continuous yield of such stratum. A test well and one or two observation wells should be driven and the supply studied from

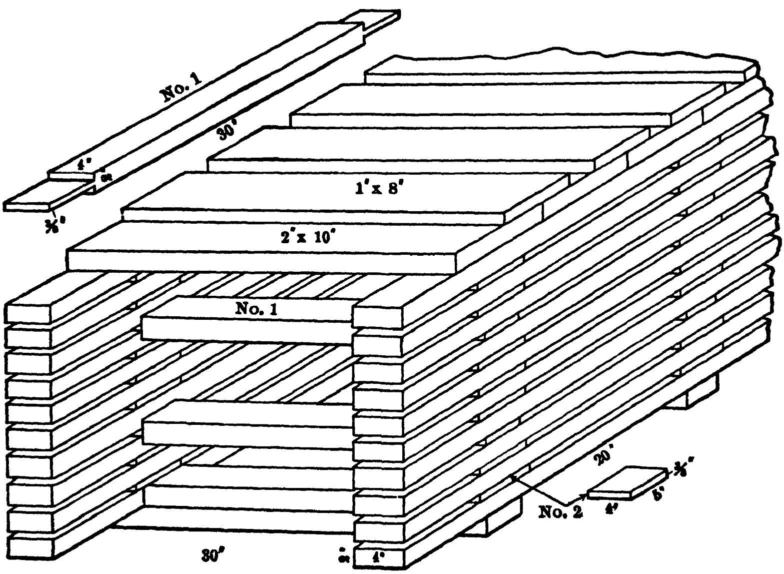


FIG. 96.—Infiltration Crib, Denver Water Works.
(From Trans. Am. Soc. C.E., Vol. XXXI.)

these when the wells are shallow; but if the supply lies deep, and there are no other wells in the vicinity to judge by, such observation wells would be too expensive, and one service well should be driven, each water-bearing stratum encountered being tested by pumping at different rates and noting drop in ground-water surface at each rate, and rate of rise in level when pumping stops. From these data an opinion must be formed as to desirable size, depth, number, and spacing of wells, and the layout of wells and location of pumping station planned accordingly.

Infiltration cribs and galleries are practically elongated

wells roofed over below the surface. Timber ones are ordinarily made on the general plan shown in Fig. 96. They should be used only where the entire crib is always immersed, as otherwise they are liable to decay. A masonry gallery may be made in the form of a circular sewer, with a great number of openings left in the walls; or may have vertical walls and an arched top, the bottom being open to admit the water. Brooklyn, N. Y., has as an infiltration gallery about six miles of 20 to 36-inch vitrified sewer pipe laid with open joints 10 to 15 feet below normal ground water level and surrounded with coarse gravel. Infiltration galleries and cribs generally lead directly to or are connected by pipe with a suction well, from which the water is pumped. They may be extended from time to time as the demand increases, always crossing the line of ground-flow approximately at a right angle. A manhole should be provided to give access for repairs and cleaning.

Art. 79. Purification Plants

Decision as to the kind of purification necessary should be based on careful consideration of the principles stated in Chapter IV, in the light of information concerning the water obtained by physical, chemical and bacterial analyses and a thorough sanitary survey of the watershed, combined with knowledge of the geological formations from which it is obtained, if from underground sources. The various analyses should be made at intervals throughout at least a year, to determine both qualitatively and quantitatively the impurities under various conditions of high water, drought, heat, cold, etc. Most large cities have conducted, during such a period of testing, a series of experiments in purification of the water in question in small plants in order to determine exactly what methods, proportions, and details of purification plant will be most effective and economical. Without at least a long-time series of analyses, the designer cannot be sure that any given plant will be the most satisfactory possible for the water in question. Every waterworks superintendent should have such analyses made periodi-

cally of the water he furnishes, even though purification may not be contemplated for many years; and then, when it is decided on, these data may not only save time, but will undoubtedly make possible a wiser decision than would otherwise be possible.

Having decided on the general features of the plant, a definite location for it and arrangement of its several units is the next problem, to be followed by the planning of the details.

If the system is a gravity one, it will seldom if ever be possible to place the purification plant above the impounding reservoir without pumping, the expense of which is to be avoided if possible. If a pressure filter is to be used, this may be placed at any point along the main pressure conduit, although the nearer the reservoir the better. If there is a distributing reservoir, the best place for the plant is generally between the impounding and distributing reservoirs, preferably near the latter for convenience, but the land available will probably determine the site. A filtration plant requires a fairly level area, except for small rapid sand filters which can all be placed in one small building. No sedimentation reservoir is needed, the impounding reservoir serving that purpose; but a coagulating reservoir must be provided. If there is not a distributing reservoir below the plant, a "clear-water basin" must be provided of capacity sufficient to provide for daily variations in consumption, since the rate of filtration should remain fairly constant throughout the day. If the impounding reservoir feeds directly into the distribution system and it is not desirable to reduce the head more than 10 feet or so, the plant must be placed at some point just below this reservoir, or at least at an elevation just below the level of low water in it; or else the plant may be placed slightly above the reservoir and water pumped from the latter to the filter. The best solution of this problem of location will be different for almost every case.

If the system is a pumping one, the water being pumped direct to the reservoir, the purification plant may be placed at and slightly higher than the reservoir and the water be pumped directly to it. But if the system is a direct-indirect one, the

water must be purified before it enters the pumps; and this is always preferable because it permits economy by combining the purification and pumping labor force, it lessens the wear on the pumps by giving them clearer water to pump, and it avoids danger due to possible tapping of the pumping main for a supply. In this case the water must generally be lifted 10 to 30 feet from river or lake into a sedimentation basin, and this lift should be minimized by placing the plant as low as possible. (In some cases the water can be led by gravity to the plant from a point higher up stream.) But there must be certainty that the plant will not be flooded by high water; which is sometimes provided by building a dike or levee around it.

Both low-lift pumps to sedimentation basin, and high-lift pumps to distribution system, should preferably be in the same building, the former drawing from the river or other source of supply, the latter from a clear-water basin. Iron pipe is comparatively cheap and this arrangement presents no difficulties, whatever the arrangement of the several units of the plant. In some cases river or lake water flows by gravity to a low-set sedimentation basin, and the low-lift pumps draw from this. This is especially desirable when the river carries much sand, as it saves the low-lift pumps from being cut by the grit; but provision must be made for continuing operation during the highest water.

Chemical solutions are difficult to handle in pipes, and the shorter these can be made the better. Therefore the chemical mixing and applying apparatus should be located as near as possible to where most of the chemical is to be applied; and it is generally desirable to place the laboratory and superintendent's office in the same building. It is preferable that the pumping apparatus be in a separate building to avoid the jarring of the laboratory appliances by machinery, and the passing of coal dust, ashes, etc., and of heat from boiler room to laboratory. Both boiler room and chemical building must be accessible to teams, or better still to a railroad siding, to permit supplying coal and chemicals. The rest of the arrangement will be decided largely by the topography.

At the present time, except for the windows and roof of the enclosing building, practically all the structural features of a purification plant are made of concrete. These include sedimentation tanks, filters, sand bins, wash-water tanks, and conduits and galleries, foundations and walls of buildings. Wood should not be used about the plant unless possibly in cypress wash-water tanks, or for small rapid filters. Steel should always be free for inspection and not buried in the ground, although if thoroughly embedded in concrete it is free from corrosion. Cast iron is comparatively free from serious rusting or corrosion by most of the chemicals used in purification. Tanks for acid coagulant should be of concrete or iron lined with enamel, and the pipes of rubber, glass, earthenware, or certain acid-resisting metals, the best of which for sulphate of alumina is phosphor-bronze. For alkaline solutions, the tanks may be of iron or concrete, and the pipes of iron. For bleach, concrete or pure wrought iron may be used, but not wood, copper, brass or steel; the pipes may be of lead, glass, stoneware, pure wrought iron, or acid-proof bronze.

All basins should be water-tight. If those built in the ground are not, there may not only be loss of water, but this may soften the soil beneath and permit settlement; or impure ground water may enter the porous wall and pollute the water. Angles and junctions of walls give the most trouble, and it is well to reinforce these points with rods extending several feet into each wall. In the case of long concrete walls in most recent plants, expansion joints are provided at intervals; and such joints are placed in the bottom also, the joints described in Chapter XIII being used. It is not so essential that sedimentation basins be water-tight, and they are usually quite large; consequently they are often made of earth embankments with sloping faces, but these and the bottoms should be covered with concrete to permit ready removal of the sediment. It is generally desirable to provide two or more basins, and this is effected by placing dividing walls in one large basin; these walls being of reinforced concrete rather than of earth, to economize space.

Slow sand filters are generally roofed over, in which case the

construction is similar to that of covered reservoirs. Offsets in walls and columns about 18 inches above the floor are sometimes used to prevent the water reaching the bottom without passing through the sand. A drainage system is built in the bottom to remove the filtered water. This consists, in each filter bed, of a main drain below the floor, to which slopes a pipe lateral laid in each row of the inverted arches which form the bottom. The area of each bed is made between .25 acre and 1 acre, depending upon the capacity of the entire plant; it generally being desirable to have at least four beds, of which any three can purify the maximum demand while the fourth is being cleaned. A weir is provided in each bed at the highest level of the sand, over which the entering water flows at low velocity so as not to disturb the sand. Provision must be made for removing and replacing sand; either an inclined floor leading from a wide entrance at ground level, down to the sand level, or pipes for use as sand ejectors.

In addition to the filter beds, there must be provision for storing and washing sand, water must be obtainable under pressure for the latter purpose; and an office and a chemical and bacterial laboratory are almost essential. Provision should be made for taking samples of water at the outlet of each filter bed and of the unfiltered water; also for measuring the flow from each bed and the height of water on each.

Rapid sand filters are more complicated in their arrangement, especially of the piping or other conduits, since there is generally a greater number of filters, and each must have its own pipe for admitting unfiltered water and filtered wash water (also air, when this is used in washing), and for removing both filtered and wash water. Certain of the details ordinarily used are patented, and in most cases, especially in the smaller plants, the entire plant and equipment is contracted for with one of the companies making a business of filter construction. In a few cases, however (e.g., Baltimore, Md.), the entire plant has been designed by the city and let out by contract like any construction work. Owing to the unusualness of this, however, no effort will be made to describe the designing of rapid filters.

The contract should define the efficiency required in terms of guaranteed bacterial content, turbidity, color, hardness, etc., of the effluent; and specify how elaborate must be the operating mechanism, observation and recording gages, sampling pumps, rate controllers, chemical mixing and feeding contrivances, etc. For a small plant, simple hand-operated contrivances can be used for all these purposes; but for a large and elaborate one, hydraulic valves, automatic rate-of-flow controllers, automatic chemical feeds, and devices recording every action of each bed are generally provided. These add considerably to the cost, but enable one man to operate a much larger number of beds and ensure more certain efficiency of filtration than would otherwise be possible.

The number of beds generally lies between four and twelve, and the practice is to arrange them in two rows with a pipe gallery between. The filters may be entirely housed over, or only a few feet of the ends next to the pipe gallery may be inside the building, the rest being outside the building but covered with a flat roof a few feet above the sand. The pipe gallery is generally 6 to 20 feet wide, the width depending largely upon the size of pipes. Each line of beds extends 20 to 50 feet from this gallery; making the extreme width, including walls, 60 to 125 feet. As about 350 square feet of sand area is provided for each million gallons per day, the total area of beds may be calculated. Then, allowing for walls, the area required to accommodate the filter can be calculated. The clear-water basin is sometimes placed directly under the filters, in other cases adjacent to them. There must also be a coagulating basin, and in some cases a sedimentation basin also. For the sizes of these, see Chapter IV.

ART. 80. CONDUITS AND DISTRIBUTION SYSTEMS

Canals of earth construction embody the same principles as do earth dams and reservoirs. The side slopes are generally about 1½ to 1, the tops of embankments 6 to 12 feet wide. The same precautions and means are taken to prevent leakage. Canals with masonry walls may be used with satisfaction

where bed-rock is found at the surface. This will generally be on the side hills, where the canal will be formed on a bench. The rock will then form the bottom and usually one side, and the other side is formed by a concrete or masonry wall, the stone for which is obtained in the excavation. The wall should be at least 2 or 3 feet thick on top, and of trapezoidal section. For masonry it is better to use small "one-man" stones, laid in rich cement mortar thoroughly filling all joints. If the rock is at all seamy or porous, the canal should be lined with concrete and given a cement coating.

Another form of open conduit is the flume, made wholly or partly of timber, or of steel. A rectangular flume is shown in

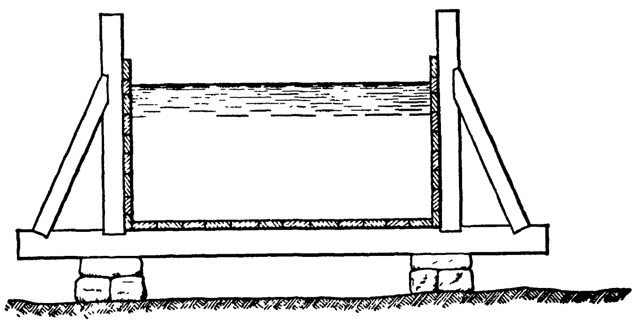


FIG. 97.—Box Flume Construction.

Fig. 97. A tighter and more durable flume, known as the stave and binder flume, has been used in the West. Its general construction is shown in Fig. 98. In this the bottom is made like the lower half of a wood-stave pipe, but vertical sides take the place of a closed top; a binding-rod passes around the flume, its ends passing through the two ends of a cross-head and being provided with nuts by which the staves are forced together. This flume is supported by stiff U-shaped frames of T iron resting on wooden bolsters or sills and spaced 8 feet apart, each frame resting on concrete foot-blocks.

Flumes resting on benches should be supported wholly by the original soil, and in no place by filling. The sills should be raised above the ground a short distance and supported on solid stone, brick, or concrete foot-blocks. The excavation,

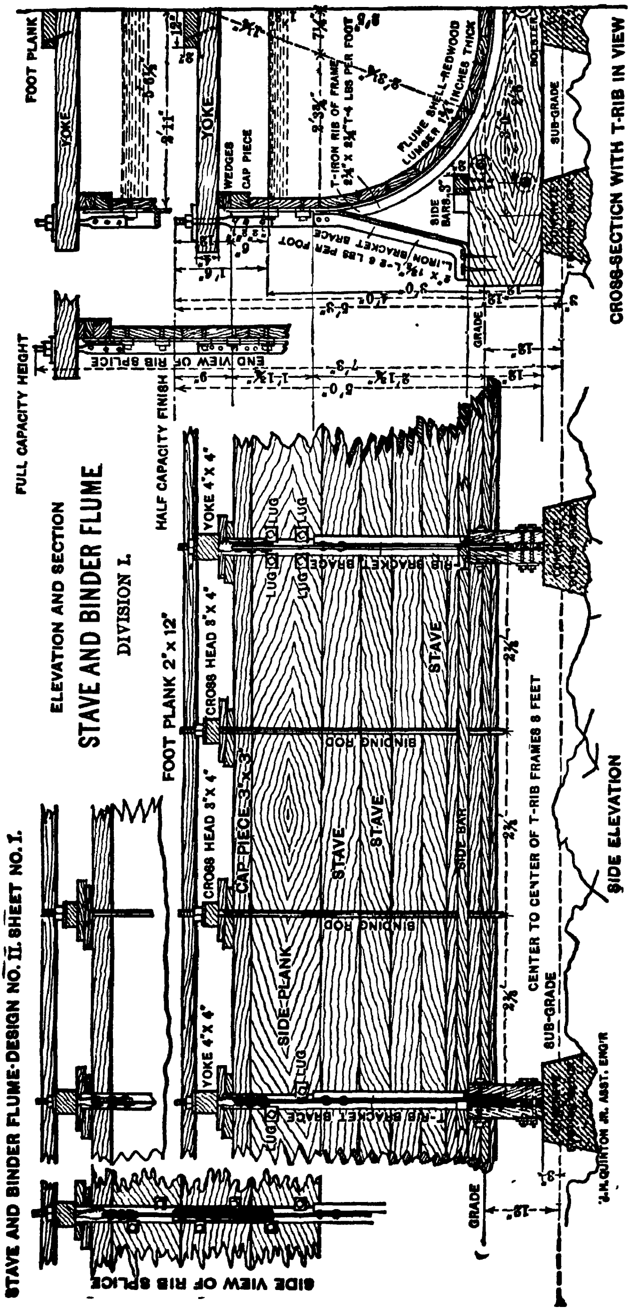


FIG. 98.—Stave and Binder Flume.

if any, should be carried into the hillside 3 or 4 feet beyond the flume, to lessen the danger of this being damaged by stones or earth falling from above. The timber used should be that which is least liable to decay.

Creosoting will add considerably to the life of the timber. Or if all surfaces intended to come in contact are first painted with hot asphaltum or tar, decay will be considerably delayed.

Flumes are also built of steel plates shaped like the stave and binder flume and riveted together, supported in steel yokes.

Flumes of all kinds are carried across valleys on trestles; or the valley may be crossed by a pressure conduit connected to a flume at each end. A flume must of course follow the hydraulic gradient of the line.

A closed gravity conduit may be made by covering a flume with plank, or may be constructed of masonry. The latter plan is generally adopted, and the conduit is also covered with a few feet of earth to protect it from frost in winter and heat in summer, and from malicious damage. It is generally circular in section when in firm soil, this shape being the most economical. It may be made of brick, stone, or concrete. If of brick, two concentric rings are generally employed up to a diameter of about 5 feet; three rings up to about 10 feet. When on the surface, or on a timber or other foundation, the "horseshoe" section is ordinarily employed, having an arched or flat top, vertical side walls sufficiently heavy to receive the thrust of the arch, and a flat inverted arch at the bottom.

The foundation of a masonry conduit or aqueduct must be absolutely rigid, since the least settlement and crack in the aqueduct may lead to serious results. To prevent softening of the supporting soil, as well as waste, the masonry should be water-tight. The Wachusett, Mass., aqueduct, where crossing a valley on a masonry bridge, was rendered water-tight, in spite of any slight settlement of the bridge arches, by lining the channel with sheet lead weighing 5 pounds to the square foot, and protecting this from wear by an inside layer of brick.

Conduits under pressure cannot be built of stone masonry, but a few have been built of reinforced concrete. (At Victoria,

B. C., 27 miles of 42-inch reinforced concrete conduit was constructed, to carry a maximum pressure of 39 pounds per square inch.) Large pressure conduits are sometimes carried through rock in tunnel, generally lined with concrete to reduce friction and prevent leakage or seepage into the conduit.

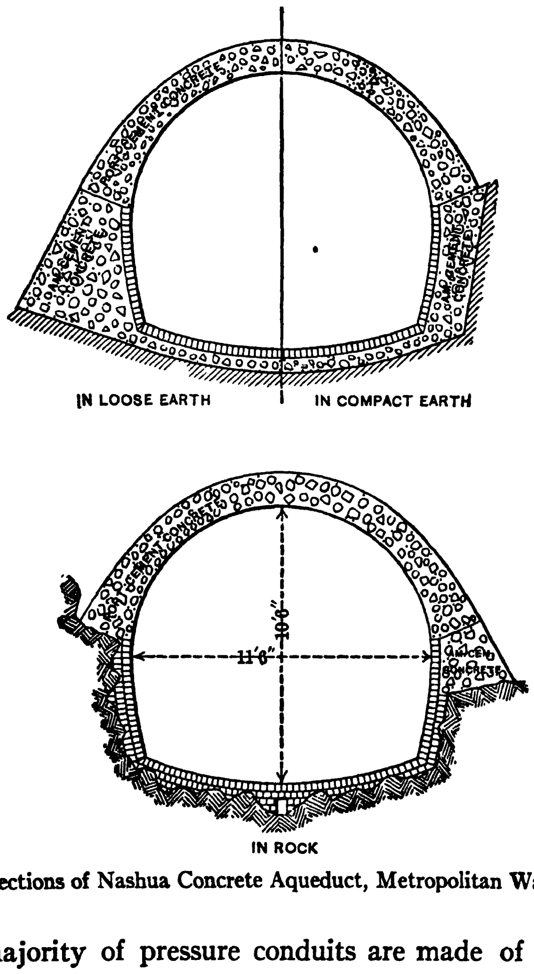


FIG. 99.—Sections of Nashua Concrete Aqueduct, Metropolitan Water Supply.

The majority of pressure conduits are made of cast iron, steel, or wood, the last either as bored logs or stave-pipe. In the metal pipes the pressure is resisted by the tensile strength of the material itself. Bored logs exert this to a certain extent; but if to sustain much pressure, they are tightly wrapped with

wire or iron bands. Stave-pipes rely entirely upon iron bands for their resistance to internal pressure; except that between these the stiffness of the staves prevents springing and leaking.

The pressure to be resisted by a pipe—that is, the resultant component of all forces acting in any one direction in its cross-section—equals $WhdL$, in which W is the weight of a cubic foot of water, d is the diameter of the pipe, h is the head upon the center of the pipe, and L is the length of pipe considered. If we make L 1 inch, and express d in inches, making W the pressure upon a square inch due to one foot head, the internal pressure $P = .434hd$ pounds per lineal inch. This is resisted equally (in a circular pipe) by both sides, and hence the tension per square inch of metal $T = \frac{.434hd}{2t}$, in which t is the thickness of the shell in inches. h must be taken as the maximum head possible, including that due to water-hammer; and a sufficient factor of safety must be used, which will vary with the substance employed. A minimum thickness must be adopted, below which the pipe would be subject to distortion or breakage by handling. If we allow 200 pounds for water-ram and call s the ultimate tensile strength of the substance used and F the factor of safety, the formula for thickness will be $t = \frac{(.434h + 200)dF}{2s}$. This formula does not apply to pipe where the tension is sustained by bands. In these $P = .434hd$ pounds per lineal inch and the tension in each rod is $\frac{.434hdb}{2}$, in which b is the distance between bands in inches. This tensile strain is of course acting in every part of either pipe or band.

The minimum limit for thickness of cast-iron pipe may be taken to be:

for 4-inch diameter.........	$\frac{3}{8}$ in.	for 24-inch diameter........	$\frac{11}{16}$ in.
for 8-inch diameter.........	$\frac{7}{16}$ in.	for 30-inch diameter........	$\frac{3}{4}$ in.
for 10-inch diameter........	$\frac{1}{2}$ in.	for 42-inch diameter........	$\frac{13}{16}$ in.
for 16-inch diameter........	$\frac{1}{2}$ in.	for 48-inch diameter........	$\frac{7}{8}$ in.
for 20-inch diameter........	$\frac{5}{8}$ in.	for 60-inch diameter........	$1\frac{1}{4}$ in.

The tensile strength of ordinarily good cast iron may be taken at 18,000 pounds per square inch. Many formulas

have been advanced for determining the thickness of pipe, allowing for minimum thickness and tensile strength. That of James B. Francis is

$$t = .000058hd + .0152d + .312;$$

in which t and d are in inches and h is in feet

$$t = .00006hd + .0155d + .296$$

is used by the Warren Foundry and Machine Company.

Cast-iron water pipes are ordinarily connected by hub-and-spigot joints, filled with melted lead which is " set up " with calking-tools; with melted " leadite " (a sulphur compound that is not calked), or in some cases with other substances. A joint known as the " Universal " is coming into increasing use,

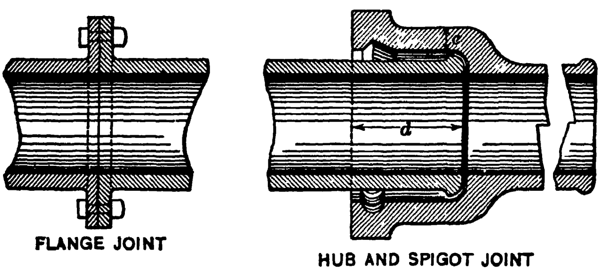
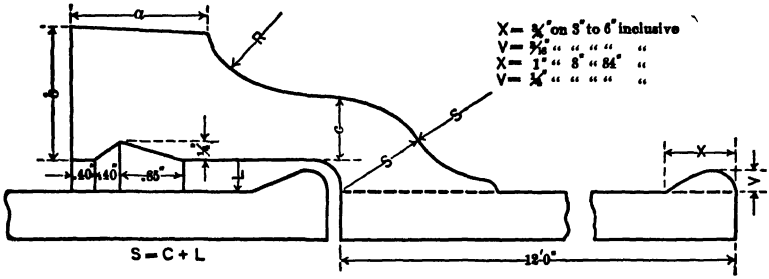


FIG. 100.—Joints of Cast-iron Pipe.

being a machined taper joint that is drawn together by bolts and depends on perfection of contact alone for water-tightness. For indoor work, for joints which it is desired to unmake occasionally, and some other conditions, flanged pipe is used; but it is not desirable for underground work because of its inflexibility, and is more expensive than the hub-and-spigot joint. The flanges of flanged joints are faced down to a true plane at exact right angles to the axis of the pipe; and in joining them a ring of packing—sheets of rubber and cotton, corrugated copper, lead wire, and other materials are used, the first being the most common—is placed between the flanges, which are drawn tightly together by bolts.

A standard form of bell and spigot and standard thicknesses of cast-iron pipe for different diameters and different pressures

have been adopted by the American Water Works Assocation and are in quite general use. Eight classes are provided, called *A, B, C, D, E, F, G* and *H*, and designed for maximum heads of 100, 200, 300, 400, 500, 600, 700, and 800 feet respectively. The dimensions and weights of these standards are given in the following tables:



Standard Form of Bell and Spigot.
The letters *a*, *b* and *c* in the table refer to this diagram.

Table No. 44

A. W. W. A. STANDARD DIMENSIONS OF PIPE

Nominal Diam. Inches.	Classes.	Actual Outside Diam. Inches.	Diam. of Sockets.		Depth of Sockets.		*a*	*b*	*c*
			Pipe, Inches.	Special Castings, Inches.	Pipe, Inches	Special Castings, Inches.			
4	A	4 80	5 60	5 70	3.50	4 00	1 5	1 30	.65
4	B–C–D	5 00	5.80	5 70	3.50	4 00	1 5	1 30	.65
6	A	6 90	7.70	7.80	3.50	4.00	1 5	1 40	.70
6	B–C–D	7.10	7.90	7.80	3.50	4 00	1 5	1 40	.70
8	A–B	9 05	9.85	10.00	4.00	4.00	1.5	1 50	.75
8	C–D	9 30	10.10	10.00	4.00	4 00	1.5	1 50	.75
10	A–B	11.10	11.90	12.10	4.00	4.00	1.5	1.50	.75
10	C–D	11 40	12.20	12.10	4.00	4.00	1.5	1.60	.80
12	A–B	13.20	14.00	14.20	4.00	4.00	1.5	1.60	.80
12	C–D	13.50	14.30	14.20	4 00	4.00	1.5	1 70	85
14	A–B	15.30	16.10	16.10	4.00	4.00	1.5	1.70	.85
14	C–D	15 65	16.45	16.45	4.00	4.00	1.5	1.80	.90
16	A–B	17 40	18.40	18.40	4.00	4.00	1.75	1.80	.90
16	C–D	17.80	18.80	18 80	4 00	4.00	1.75	1.90	1.00
18	A–B	19.50	20.50	20.50	4 00	4 00	1.75	1.90	.95
18	C–D	19.92	20.92	20.92	4 00	4.00	1.75	2.10	1.05

TABLE No. 44—*Continued*

Nominal Diam. Inches.	Classes.	Actual Outside Diam. Inches.	Diam. of Sockets.		Depth of Sockets.		*a*	*b*	*c*
			Pipe, Inches.	Special Castings, Inches.	Pipe, Inches.	Special Castings, Inches.			
20	A–B	21.60	22.60	22 60	4.00	4.00	1.75	2 00	1 00
20	C–D	22.06	23.06	23.06	4.00	4.00	1.75	2 30	1 15
24	A–B	25.80	26.80	26.80	4.00	4.00	2.00	2 10	1.05
24	C–D	26.32	27.32	27.32	4.00	4.00	2.00	2 50	1 25
30	A	31.74	32.74	32.74	4 50	4.50	2.00	2.30	1 15
30	B	32.00	33.00	33.00	4.50	4.50	2.00	2 30	1.15
30	C	32 40	33.40	33.40	4.50	4.50	2.00	2.60	1 32
30	D	32 74	33.74	33.74	4.50	4.50	2 00	3.00	1 50
36	A	37.96	38.96	38.96	4 50	4.50	2.00	2.50	1 25
36	B	38.30	39.30	39 30	4 50	4 50	2.00	2 80	1 40
36	C	38.70	39.70	39.70	4 50	4 50	2.00	3 10	1 60
36	D	39.16	40.16	40.16	4.50	4.50	2.00	3 40	1 80
42	A	44.20	45.20	45.20	5 00	5 00	2 00	2.80	1 40
42	B	44.50	45 50	45.50	5 00	5 00	2.00	3.00	1.50
42	C	45.10	46.10	46 10	5 00	5.00	2.00	3 40	1 75
42	D	45 58	46.58	46.58	5.00	5.00	2.00	3 80	1 95
48	A	50 50	51.50	51.50	5.00	5.00	2.00	3 00	1.50
48	B	50.80	51.80	51.80	5.00	5.00	2 00	3.30	1 65
48	C	51.40	52.40	52.40	5 00	5 00	2 00	3.80	1.95
48	D	51 98	52.98	52.98	5.00	5.00	2.00	4 20	2.20
54	A	56 66	57 66	57.66	5 50	5 50	2.25	3 20	1 60
54	B	57 10	58 10	58 10	5 50	5 50	2 25	3 60	1 80
44	C	57.80	58.80	58.80	5 50	5.50	2.25	4.00	2.15
54	D	58.40	59 40	59 40	5 50	5 50	2 25	4 40	2 45
60	A	62.80	63.80	63.80	5.50	5.50	2.25	3.40	1.70
60	B	63.40	64 40	64.40	5.50	5.50	2.25	3.70	1 90
60	C	64.20	65.20	65.20	5.50	5.50	2.25	4 20	2.25
60	D	64.82	65.82	65.82	5.50	5.50	2.25	4.70	2 60
72	A	75.34	76.34	76 34	5.50	5.50	2.25	3.80	1.87
72	B	76.00	77.00	77.00	5 50	5.50	2.25	4.20	2.20
72	C	76.88	77.88	77.88	5.50	5.50	2.25	4.60	2.64
84	A	87.54	88 54	88 54	5.50	5 50	2.50	4.10	2.10
84	B	88.54	89.54	89.54	5 50	5.50	2.50	4.50	2.60

All pipes and special castings should be made of cast iron of good quality, and of such character as will make the metal

of the castings strong, tough and of even grain, and soft enough to admit of drilling and cutting satisfactorily. The metal should be made without any admixture of cinder iron or other inferior metal, and should be remelted in a cupola or air furnace.

TABLE No. 44—*Continued*

Nominal Diam. Inches.	Classes.	Actual Outside Diam. Inches.	Diam. of Sockets, Pipe and Specials.	Depth of Sockets, Pipe and Specials.	*a*	*b*	*c*	*R*
6	E–F	7 22	8 02	4 00	1 50	1 75	.75	1 10
6	G–H	7 38	8 18	4 00	1 50	1 85	.85	1 10
8	E–F	9 42	10 22	4 00	1 50	1 85	.85	1.10
8	G–H	9 60	10 40	4 00	1 50	1 95	95	1.10
10	E–F	11 60	12 40	4 50	1 75	1 95	.95	1 10
10	G–H	11 84	12 64	4 50	1 75	2 05	1 05	1 10
12	E–F	13 78	14 58	4 50	1 75	2 05	1 05	1 10
12	G–H	14.08	14 88	4 50	1 75	2 20	1 20	1 10
14	E–F	15 98	16 78	4 50	2 00	2 15	1 15	1.10
14	G–H	16 32	17.12	4 50	2 00	2.35	1 35	1 10
16	E–F	18 16	18 96	4 50	2 00	2 30	1 25	1 15
16	G–H	18 54	19 34	4 50	2 00	2 55	1 45	1 15
18	E–F	20 34	21 14	4 50	2 25	2 45	1 40	1 15
18	G–H	20 78	21 58	4 50	2 25	2 75	1.65	1 15
20	E–F	22 54	23.34	4 50	2 25	2 55	1 50	1 15
20	G–H	23 02	23 82	4 50	2 25	2 85	1 75	1 20
24	E–F	26 90	27 90	5 00	2 25	2 85	1 70	1 20
30	E	33 10	34 10	5 00	2.25	3 25	1 80	1 50
30	F	33 46	34 46	5 00	2 25	3 50	2 00	1 55
36	E	39 60	40 60	5 00	2 25	3 70	2 05	1 70
36	F	40 04	41 04	5 00	2 25	4 00	2 30	1 80

Cast-iron pipe will become covered with rust and, with some classes of waters passing through it, with tubercles unless protected with a coating. Several substances have been tried for this, but that which has proved most satisfactory and is now in general use is known as coal-tar pitch varnish. In applying this, the pipes are immersed in a hot bath of boiling coal-tar pitch from which the naphtha compounds have been distilled off and to which a small amount of mineral oil has been added to give fluidity. Both casting and pitch should be heated to 300° when the pipe is immersed, and the pipe remain in the bath for five minutes, then be drained of superfluous varnish.

Table No. 45.—A. W. W. A. Standard Thickness and Weights of Cast-Iron Pipe

Nominal Inside Diameter Inches.	Class A 100-ft. Head, 43 Lbs. Pressure.			Class B 200-ft. Head, 86 Lbs. Pressure.			Class C 300-ft. Head, 130 Lbs. Pressure.			Class D 400-ft. Head, 173 Lbs. Pressure.		
	Thickness Inches.	Weight per Foot.	Weight per Length	Thickness Inches.	Weight per Foot.	Weight per Length.	Thickness Inches.	Weight per Foot	Weight per Length.	Thickness Inches.	Weight per Foot.	Weight per Length.
4	.42	20 0	240	.45	21 7	260	.48	23 3	280	52	25.0	300
6	.44	30.8	370	.48	33 3	400	.51	35 8	430	.55	38.3	460
8	.46	42 9	515	.51	47 5	570	.56	52 1	625	.60	55.8	670
10	.50	57.1	685	.57	63 8	765	62	70 8	850	.68	76.7	920
12	.54	72 5	870	.62	82 1	985	68	91 7	1100	.75	100.0	1200
14	.57	89.6	1075	.66	102 5	1230	74	116 7	1400	.82	129.2	1550
16	.60	108 3	1300	.70	125 0	1500	80	143 8	1725	89	158 3	1900
18	.64	129 2	1550	.75	150 0	1800	87	175 0	2100	96	191.7	2300
20	.67	150 0	1800	80	175 0	2100	.92	208 3	2500	1 03	229.2	2750
24	.76	204.2	2450	.89	233 3	2800	1 04	279 2	3350	1 16	306.7	3680
30	.88	291 7	3500	1 03	333 3	4000	1 20	400 0	4800	1 37	450.0	5400
36	.99	391 7	4700	1 15	454 2	5450	1 36	545 8	6550	1 58	625.0	7500
42	1.10	512 5	6150	1 28	591 7	7100	1 54	716 7	8600	1.78	825.0	9900
48	1.26	666.7	8000	1 42	750 0	9000	1 71	908 3	10900	1 96	1050 0	12600
54	1.35	800 0	9600	1 55	933 3	11200	1 90	1141 7	13700	2 23	1341.7	16100
60	1.39	916 7	11000	1 67	1104 2	13250	2 00	1341 7	16100	2 38	1583 3	19000
72	1.62	1283 4	15400	1 95	1545 8	18550	2 39	1904 2	22850			
84	1.72	1633 4	19600	2.22	2104 2	25250						

The above weights are per length to lay 12 feet including standard sockets; proportionate allowance to be made for any variation.

TABLE No. 45—*Continued*

Nominal Inside Diameter. Inches.	CLASS E 500-ft. Head, 217 Lbs. Pressure.			CLASS F 600-ft. Head, 260 Lbs. Pressure.			CLASS G 700-ft. Head, 340 Lbs. Pressure.			CLASS H 800-ft. Head, 347 Lbs. Pres.		
	Thickness Inches.	Weight per Foot.	Weight per Length.	Thickness Inches.	Weight per Foot.	Weight per Length.	Thickness Inches.	Weight per Foot.	Weight per Length.	Thickness Inches	Weight per Foot.	Weight per Length.
6	.58	41 7	500	61	43 3	520	.65	47 1	565	.69	49.6	595
8	.66	61 7	740	.71	65.7	790	.75	70 8	850	.80	75.0	900
10	.74	86 3	1035	.80	92 1	1105	.86	100 9	1210	.92	106.7	1280
12	.82	113 8	1365	.89	122 1	1465	.97	135 4	1625	1 04	143 8	1725
14	.90	145.0	1740	.99	157 5	1890	1.07	174 2	2090	1 16	186.7	2240
16	98	179 6	2155	1 08	195 4	2345	1.18	219.2	2620	1.27	232.5	2790
18	1.07	220 4	2645	1.17	238 4	2860	1.28	267 1	3205	1.39	286 7	3440
20	1.15	263 0	3155	1.27	286 3	3435	1 39	320.8	3850	1.51	344 6	4135
24	1 31	359 6	4315	1.45	392 9	4715						
30	1.55	521 7	6260	1.73	585.4	7025						
36	1.80	725.0	8700	2 02	820 0	9840						

Table No. 46

WATER MAINS, FIRE HYDRANTS AND METERS *

Size of Cities.	Population Served per Mile of Main.	No. of Fire Hydrants per Mile of Mains.	Per cent of Water Consumption Metered.
Cities of over 500,000 population..	1031	11.3	26
Cities of 300,000 to 500,000 population..	619	8.1	61
Cities of 100,000 to 300,000 population..	556	8.3	54
Cities of 50,000 to 100,000 population..	555	7.3	50
Cities of 30,000 to 50,000 population..	408	6.8	54

* Compiled from report of U. S. Census Bureau for the year 1915.

Table No. 47

RELATIVE AMOUNTS OF DIFFERENT SIZES OF WATER MAINS IN UNITED STATES CITIES *

	Total Length, Miles.	3-inch and Under. %	4-inch. %	6-inch. %	8-inch. %	10-inch %	12-inch %	14-inch and Larger. %
Laid during the year 1915, averages of 389 cities........	857	5 7	15 9	41.0	13 9	5 4	7 8	10 3
Total in service in 1916, averages of 585 cities.	22,286	5 8	17 2	43 5	13 8	4 6	7 7	7 4

* Compiled from data collected by "Municipal Journal."

Cast-iron pipe is now used almost universally in this country for distribution systems, and also for small conduit lines and for some large ones up to 5 feet diameter. House connections are made with it at any point by tapping a hole in the pipe and screwing in it a corporation cock. Larger connections, such as branch lines, fire-hydrants, blow-offs, etc., are made by means of special castings.

Wrought iron or steel in thin plates is used in several ways for water-pipe. A small pipe may be made of a single plate whose edges are riveted together or are welded; or by winding a long narrow plate spirally and riveting or welding the spiral lap. Larger pipe is riveted, with lap- or butt-joints, and made

in sections 15 to 30 feet long, each formed of two or more plates; either every second section of lap-joint pipe being made smaller

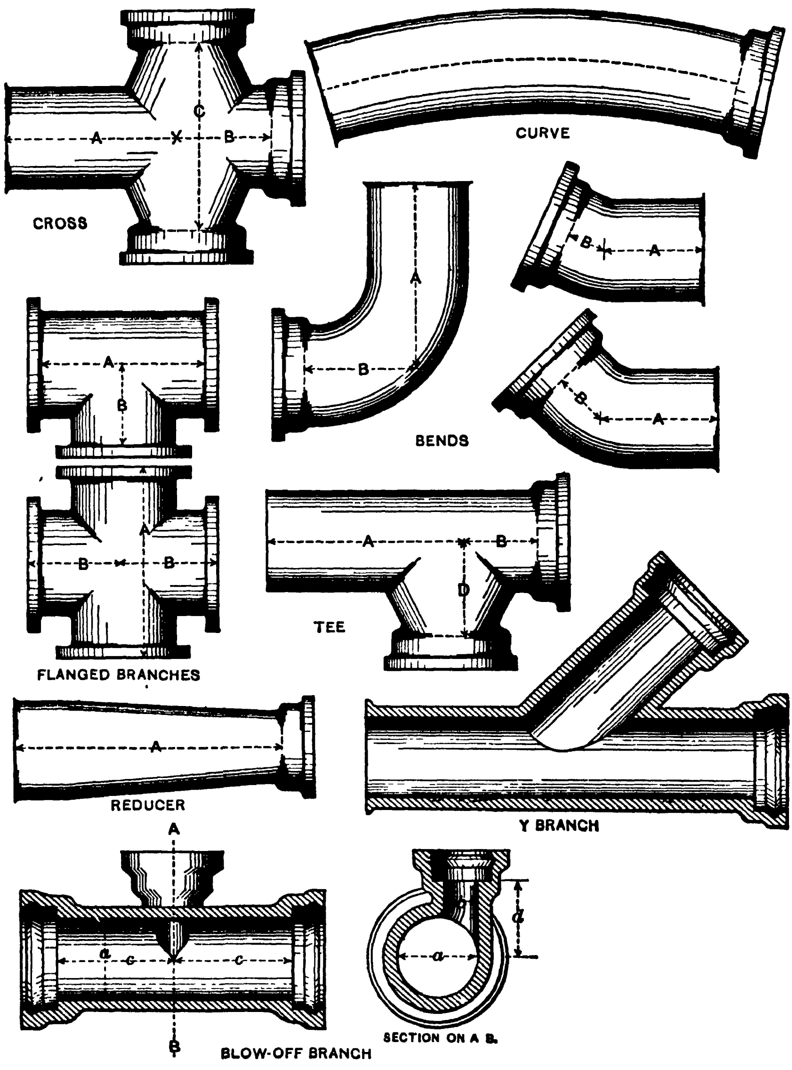


FIG. 101.—Cast-iron Pipe Specials.

than the others and fitting tightly into their ends, or each pipe is slightly tapering, one end forming the inside, the other the

outside, of a lap-joint. Another method of uniting the sheets that originated in Australia and is now used in this country is by the lock-bar; a double-grooved bar receiving the edges of the sheet and binding them tightly together under hydraulic pressure. For small pipe the lap-weld is most used; and for large, the riveted lap-joint. The pipe is generally made in 15- to 30-foot sections in the shop, and these riveted together in the field. For convenience of joining, small wrought-iron pipe is sometimes furnished with a cast-iron bell at one end and a narrow strap for a bead at the other, the pipe being joined with lead

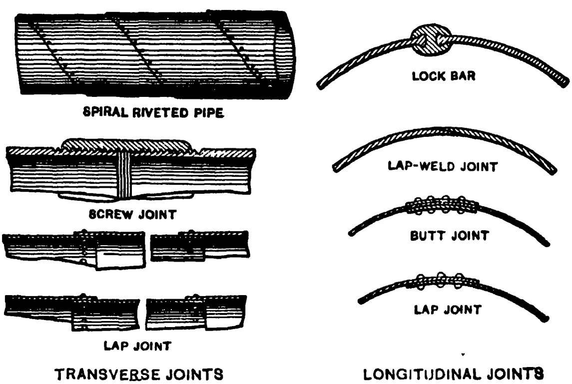


FIG. 102.—Wrought-iron Pipes.

as is cast iron. Most small pipe, however, is joined by screw couplings.

Wrought-iron or steel pipe is made much thinner than cast iron, both because the tensile strength of the metal is greater and because it can be made more uniform in character and thickness of metal and can be carefully inspected in the sheet. Also much lighter pipe can be handled without danger of breaking. The thickness is determined as for cast iron, but a lower factor of safety is used. Because of the difference in structure and thinness of the metal, this pipe rusts through much more quickly than cast iron, and hence a protective coating is even more necessary. Large wrought-iron or steel pipe is distorted by the weight of the back-filling unless the earth against

the bottom and sides be compactly rammed and the pressure on the top be evenly distributed, as by sand or gravel.

Changes of direction in riveted pipe are made either by use of short sections with converging or beveled end planes, or by means of iron castings. Branches, reducers, and all other special constructions are made of riveted plates in large pipes; and of riveted plates, but more often of iron castings, in small pipes; special designs being generally made for each case in the riveted work, but the castings used being standard, as for cast-iron pipe.

The metal should be tough and elastic, capable of shop and field riveting without any injury, and have a tensile strength of between 55,000 and 65,000 pounds, an elastic limit of not less than 30,000 pounds, and an elongation of $22\frac{1}{2}$ per cent in 8 inches. Cold-bending, punching, and similar tests also are often called for. Steel from 0.1 inch to $1\frac{1}{4}$ inches, or perhaps even thicker, has been used for riveted pipe.

For coating steel pipe several preparations have been used, many having asphalt as a principal ingredient. Rochester, N. Y., has found a mixture of equal parts of refined asphalt and refined coal tar satisfactory. Asphalt dissolved in bisulphide of carbon ("P. & B.") and other compounds have been found satisfactory in other cities.

Wrought-iron pipe weighs about 10.1 pounds per lineal foot for each foot of circumference and each quarter-inch of thickness (the width of laps being added to the pipe circumference and length); and steel about 10.2 pounds for the same unit dimensions. Or, weight of steel pipe,

$$W=40.8t(3.1416d+l)(L+l),$$

in which t is the thickness of metal in inches, d is the diameter of pipe in feet, L is the length of pipe, and l is the lap at either longitudinal or circular joints.

Riveted steel pipe is generally cheaper than cast iron for sizes larger than 30 to 36 inches, but not for smaller sizes. It is not often used in distribution systems, one serious objection

being the difficulty of making service connections to such a thin shell.

Steel pipe is adapted for bridge crossings and other locations subject to vibrations or slight movements that tend to start leaks in lead joints.

Small galvanized steel pipe, 2 inches diameter or even less, has been used for distributing pipes where only a few dwellings were to be served and no fire protection to be afforded.

Wrought-iron pipe coated inside and out with cement has been used, but its use has been practically abandoned. By its use the tensile strength and uniformity of sheet metal is obtained, the cement coating protects it from rust and gives it stiffness, and does not impart a taste to the water as does the tar coating used on cast-iron pipes. But cast-iron pipe many times as strong as the wrought iron now costs less laid in the line, is less liable to damage from handling, is more easily laid, and service connections can be made with it more readily. In many cases the cement-lined pipe has been entirely destroyed by minute cracks in the cement, which would permit rust to form in the iron and, if on the outside, roots to enter and peel off the cement from the outside of the pipe. However, if the cement lining is put on properly and good materials used, this pipe has some advantages over cast iron, particularly where there is much trouble from tuberculation.

Wood pipe offers certain advantages over iron, particularly where the cost of the latter material or of transportation is excessive. It is also free from injury by electrolysis. If the pressure is at all high, however, the pipe must depend upon iron or steel bands for strength; and the pressure must be less than that which will cause the bands to crush the wood fibers. The life of the wood, if it be always saturated, should be indefinite, and that of the pipe therefore depends upon the bands.

Wood pipe, of bored logs for small sizes or of staves banded with iron bands for larger ones up to 48 inches and from 4 to 12 feet long, are used in the distribution systems of a number of small cities. It has been made to stand a pressure of 170

pounds with safety. The joints are made with tenon and socket. Iron castings are used for branches and other large connections, while for service connections, corporation cocks with lag screw threads are screwed into the pipe.

Wood-stave pipe has been used for many years for large conduits, such as flumes for water-wheels in New England and for hydraulic mining in the West. But during the past thirty years it has come into extensive use for water-supply conduits. It has been used for pressures up to 85 pounds when made of redwood or Douglas fir; the limit depending upon the ability of the wood to resist crushing. The life of the wood should be considerable, although where exposed to the air in a dry climate evaporation keeps the outside nearly dry and decay sometimes occurs. The steel bands should be coated with protective paint, and if possible should be repainted whenever necessary. But when the pipe is buried the bands cannot be repainted. To enable them to resist rust as long as possible they should be of round rather than of flat metal; when their life should be at least fifty years in ordinary soils.

The pipe is made of a number of staves of variable length having radial edges and concentric faces, which staves are held together as a pipe by metal girth bands, usually round in section, the interval between bands being governed by the internal pressure in the pipe. The staves must be thin enough to secure complete saturation and to bend readily if the pipe is built to a curve, but must be thick enough to prevent undesirable percolation through them. The bands must be so spaced that the length of stave between bands will not bulge out in the slightest under water pressure and that the pressure will not be sufficient to crush destructively the fiber beneath the band. The staves generally run from about 1½ inches thick for 30-inch pipe to 2¾ inches for 72-inch, and the bands from ½ inch diameter for the smaller pipe to ¾ inch for the largest. The tensile stresses resisted by the bands are the sum of the initial stress caused by bolting up during construction, the pressure of the water within the pipe, and the swelling of the staves.

The staves are made to break joints in the pipe. The butt

joints are made by driving into kerfs sawed in the end of each stave thin steel plates about 1½ inches long and somewhat wider than the stave. The bands are cinched by fastening the two ends of each band into a cast-iron shoe, one end being provided with a thread and nut. The bottom of the shoe is made to fit the outside of the pipe. Curves are made in the pipe by simply bending the staves during construction, which is carried on in the field, the pipe being built in its final position. Small branches, air-escapes, etc., are attached by fastening castings on the outside of the pipe by means of bands.

Wood-stave pipe should not be coated on the inside, as this would prevent the saturation of the wood and permit decay.

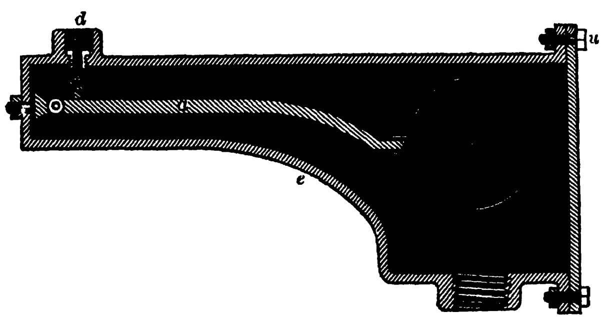


FIG. 103.—Air Escape Valve.

On all pressure conduits, blow-offs should be provided at low points for use in flushing dirt out of the pipe, and at every summit there should be an air escape. If any summit is below the hydraulic gradient, the escape must be in the form of a vertical pipe carried above the hydraulic gradient; or else so contrived as to permit air to escape, but no water, one form of which is shown in Fig. 103, *b* being a float which remains up and keeps the opening *d* closed while the box is full of water, but falls and opens *d* as air rises into the box and lowers the water. If a fire hydrant or service connection be placed at a summit, the air can escape through this and the use of an air escape valve is unnecessary. For large pipe, especially if of steel or wood stave, the air valve should be large (an open pipe carried verti-

cally above the hydraulic gradient is often used), to admit air rapidly in case of a break or other sudden discharge at a low point, which would tend to cause a vacuum at the high point. Several steel pipes have collapsed where such provision was not made.

Location. The most desirable line from a reservoir or intake to the point of utilization is a straight one, if the country be level and such a location be not expensive in right of way. But detours in a hilly country may make as short a line, or one requiring less heavy pipe, and less lift if pumping be employed; and such a detour may be necessary to avoid going above the hydraulic gradient. It is largely a question of expense, and the choice between routes should be made by comparing their cost, taking account of length of conduit, thickness of pipe necessary, accessibility and cost of right of way, special river-crossings, trestles, etc., required, and increased pumping expense or size of pipe due to lengthened line. A gravity conduit can be carried over a summit above the hydraulic gradient by use of a siphon; but this needs special attention to remove the accumulated air frequently, and is not advisable unless within easy reach of the pumping-station or a maintenance force. The air may be removed by a small vacuum-pump; or it may be collected in an air-tight tank connected with the summit of the pipe, the air in the tank, as often as it nearly fills this, being replaced with water by closing a valve at the bottom of each leg of the siphon and pouring water into the tank.

In warm climates closed conduits may be placed upon or near the surface, although this would render the water too warm for city supplies. But in cold climates this would lead to the bursting of the pipes by freezing; and they must be protected from this, generally by burying in the ground. Open conduits also in such climates should have thick walls, or earth banked against them, to prevent contraction of the channel by ice forming on the inside. Pipes through which water is always circulating are frequently left uncovered where crossing bridges, without freezing; and as a general rule it may be said that this is permissible when the length of exposed pipe in feet is not

greater than ten times the square of the diameter in inches, and the velocity of flow is at least one-fourth of a foot per second. The danger that the flow will entirely stop at nighttime is so great, however, that it is better to box in such pipe, and fill the box with mineral-wool, asbestos or some other non-conductor.

Water pipes in New England, northern New York, and the entire northern tier of states should generally be placed not less than 5 feet below the surface; south of these, to the southern tier, 4½ to 4 feet is the customary depth; and in the Southern states 3 or even 2 feet is permitted for small pipe, although when so near the surface it is in danger of being broken by heavy teams. A gravity closed conduit, if on the surface, should be covered to at least the above depth on top and sides. The above measurements are to the center of the pipe.

A pipe conduit should rest upon a firm foundation. If in rock, the bottom of the trench should be covered with gravel, sand, or loam well rammed for the pipe to be bedded on. The pipe should slope continuously (but not necessarily uniformly) to the main depressions, where blow-offs are provided discharging into a near-by creek or sewer for removing any sediment; sags in the line favoring accumulations of sediment at the low, and of air at the high, points. The blow-off may be an ordinary T, a gate valve, and a short piece of pipe leading to the sewer or creek; but it is better to substitute for the ordinary T the blow-off T shown in Fig. 101.

Curves of long radius may be made of chords connected by eighth-bends, or by making a slight angle at each joint. A 4-inch pipe may be swung 2 to 2½ feet out of a straight line, an 18-inch pipe 10 or 12 inches; and a good joint still be made. But more than this should not be attempted, as the lead joint cannot take a strong shape or be well calked if there be too much angle. Even greater angles have been made at joints of "Universal" pipe.

In crossing streams, the water-pipe may be laid in the bed of the stream or carried over on a bridge. The former plan is often expensive, and the detection and repair of leaks is

difficult. There is also danger in some cases that freshets will carry away the pipe. On the other hand, on a bridge there are vibrations tending to cause leaks, and freezing is possible; also the bridge must have additional strength to support the pipe and its contained water. Over a stone or concrete arch the pipe can be laid underground as in a street, and the only danger is that of the freezing of the pipe where the covering both above and below it is thin, as it may be over the center of the arch. Where there is no bridge, the pipe must generally be placed in the stream bed. If the pipe is carried on a bridge floor, the joints at the elbows at the top and bottom of the piece connecting the pipe on the bridge with that under ground are apt to pull apart, and should be tied

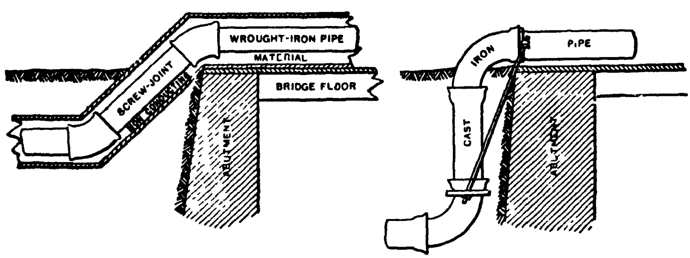


FIG. 104.—Bridge Crossings.

together by long bolts and clamp rings; or steel pipe may be substituted for cast iron across the bridge and for at least 2 or 3 feet beyond the elbows at each rising end.

Ordinarily the pipe is simply placed in the trench, the joints made, and the trench refilled with such tamping as is required by the nature of the ground or street surface. But where the pipe line passes under a railroad, a building, or other structure causing considerable pressure upon the soil, it is desirable to protect the pipe from this, and to protect the structure from being undermined by a break in the pipe. In the case of a small pipe, the latter may be effected by enclosing this pipe in a larger one. But a much better plan is to build a heavy arch of brick or concrete over the pipe, and fill the space between this arch and the pipe with sand or good earth.

For shutting off the flow at any point in a pressure conduit, gate valves with screw stems are used. Sliding-stem valves, check valves, or any kind of valve, hinged or otherwise, by which it is possible to close the opening rapidly should not be employed where the pressure is more than 10 pounds. Valves on underground pipe are reached through boxes which surround the top and stem of the valve, or in some cases enclose the entire valve, and extend to the surface, where they are closed by a cover. Valves on 12- or 15-inch pipe or less generally stand with the stem vertical, and the box is either a standard pattern of cast iron, or is built around the valve with brick. Larger valves would ordinarily rise too near the surface if erect, and are laid upon their sides; and the largest are worked by gearing. These larger valves require that a manhole be built around them, similar to a sewer manhole.

All rubbing surfaces of valves should be of bronze, and all parts sufficiently strong to resist the greatest pressure (including water-hammer) which can come upon either side of the gate. Each valve should be tested in place for leaks in the stuffing-box or elsewhere.

The location of valves and fire hydrants has already been discussed. Post fire hydrants (those extending above the surface of the street) are most commonly used. They are generally placed upon the same side of the street as is the pipe (this giving the shortest connecting pipe or branch) and just inside of the curb. The branch pipe may be 4 inches for $2\frac{1}{2}$-inch nozzles only, but if a fire-engine nozzle is furnished on the hydrant no less than a 6-inch pipe should be used. For ordinary localities two hose nozzles and an engine nozzle are sufficient. Each nozzle may be furnished with an independent valve, but this is not common. The main valve is placed at the base of the standpipe or barrel of the hydrant, and may be either a sliding gate valve, or a compression valve; and the latter may close against or with the pressure. All these have been used with satisfaction; and if the material and workmanship are good, a clear and large waterway and easy curves at the bends are more important than the style of valve. It is desirable also

that it be possible to remove the valve and stem for repairs or new " leather " without having to dig around the hydrant.

When the main valve is closed after use, the hydrant will be full of water. If this remains there in cold weather, it is apt to freeze and burst the hydrant, and hence a " drip " should be provided which will always be open when the hydrant is closed, and only then. Through this the water escapes from the hydrant, and should be led away before it can freeze. This is best accomplished by connecting to the drip a 1-inch pipe leading to a sewer or drain. If there be no drain near by, the excavation around the hydrant and branch may be filled with broken stone for a height of a foot or 18 inches, which will receive the water in its interstices and from which it will gradually soak into the ground. If the soil is any but the hardest, the hydrant should be set upon a large flat stone or a bed of concrete to prevent it from settling into the ground when this becomes wet from the drip. A large stone or small pier of masonry should be wedged tightly between the back of the hydrant opposite the branch and the solid earth to prevent the impulse of entering water from forcing the hydrant off of the branch.

If the branch leads from a main 10 to 12 inches or more in diameter, it should always be provided with a gate valve just outside the hydrant, that any injury to this may be repaired without closing the main pipe; and such valve is desirable on all hydrant branches. A frost-case is sometimes used, i.e. a cylinder of iron fitting loosely around the hydrant barrel at and for 2 feet or more below the ground surface, to prevent the " frost " from " heaving " the hydrant, which the ground surface in this case does not touch. The use of this, however, is being largely discontinued as being unnecessary.

The depth at which the branch is laid should be at least as great as that of the main, since the former is the more liable to freeze, there being no flow through it most of the time.

When placed at a corner where a large and small main intersect, the hydrant branch should generally lead from the larger pipe. Fire hydrants should not be placed directly in

front of buildings needing especial protection or which would furnish a very hot fire, but one should be preferably about 150 or 200 feet on each side of such point, as in the former position they would be inaccessible in time of fire on account of heat or falling walls.

Probably ten times as many fire hydrants are injured by sprinkling-cart drivers as in any other way, where these take water directly from the hydrants. This should not be permitted, but "water cranes"—contrivances especially designed for filling sprinkling-carts—should be placed at convenient points; and all but firemen should be forbidden, under a heavy penalty, from opening a fire hydrant. A space under each crane should be paved to catch the drip and leaking of carts, which pavement should be connected with the sewer, or with the gutter if there be no other drain.

The sizes of the main conduits leading to a city are readily calculated by the laws of hydraulics if the fall in the hydraulic gradient and the maximum quantity of flow are known. The gradient is the grade of the conduit itself if this be open. If it be closed, but if there be an open conduit or reservoir at each end and the closed conduit be uniform in bore, it is the line of uniform fall from one open end to the other.

A pressure conduit must carry, with the available head, the maximum rate per minute and still leave a sufficient pressure head at the point of supply. The gradient will then have one end at an elevation directly above the town equal to the desired pressure head, the other at a distance below the reservoir surface equal to the entrance and velocity head; the difference between these two elevations being the head available for overcoming friction in the pipe. If the pipe be continuous and of uniform bore, the gradient will be a straight line connecting these points. If a pump raise the water to the town, the end of the gradient will be at a distance vertically above the pump equal to the pressure head in the main at the pump. Thus, in Fig. 105, if A be the pressure head at the town, the line CE be horizontal, and D be a point at a distance above the pump equal to the pressure head at the pump, and the line CD will be the

hydraulic gradient, *DE* being the friction head. *D* can of course be raised to any height, and thus the gradient be given any desired fall. *A* should be at least 150 feet for cities, if possible, and 100 feet for suburban districts and villages.

The maximum rate of consumption may be taken as about 200 per cent of the average annual rate, plus the rate for fire purposes. For example, for a city of 10,000, if the pressure and nozzle size be such as to discharge a fire stream of 250 gals. per minute, 3000 gallons per minute will be the maximum amount required for this purpose; other consumption would have a maximum of, say, $\frac{10,000 \times 200}{60 \times 24} = 1400$ gals. per minute, or

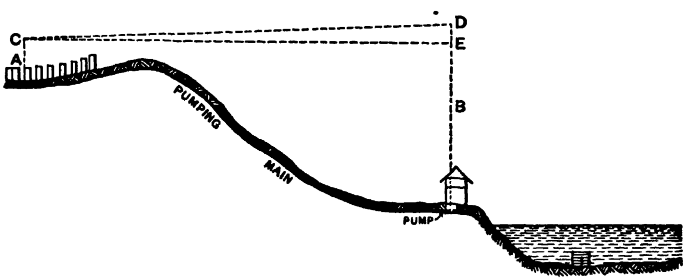


FIG. 105.—Hydraulic Gradient for Pumping Main.

a total of 4400 gals. per minute as a maximum flow to be provided for.

Calculating the sizes of the pipes required for a city distribution system is a very complex problem. If *MA*, Fig. 106, is the main conduit, and *H* a fire hydrant in service, the supply for this will come partly through each of the pipes *BI*, *AG*, and *DF*, with perhaps some flow through *KE*. *BK* will supply a large part of the discharge at *N*, and *DE* of that at *O*, although a considerable amount for each will come through *AG* and *FI*. It *might* be possible to so arrange and proportion the pipe that the supply for any one fire hydrant would come equally through all connecting lines. But the plan generally adopted is to design a skeleton system of mains and fill in intermediate streets with smaller distributing pipes. During periods of heavy draught, as when fighting fires, this gives more pressure along the

main lines than on the intermediates, and hence these lines might be placed where are the highest or most important buildings, if there be any difference in this respect. In general, the mains may pass through every second or third street running in one direction, and every fourth to eighth in the other, the remaining streets being filled with 6-inch pipe, or perhaps 4-inch in short blocks (a practice not recommended by a great many engineers); it being assumed that all the supply passes through the larger mains, which is an error on the safe side. For instance, it is assumed that in Fig. 106, *AG* carries the entire supply for the line *TU* and all the area to the right of it, or $\frac{7}{8}$ of the entire

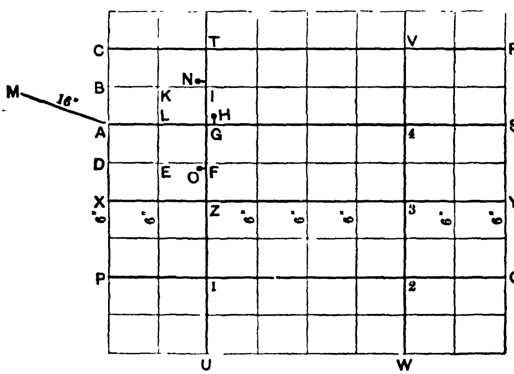


FIG. 106.—Distribution System.

ordinary consumption. For fire supply, any area greater than half the city should be provided to receive the entire supply designed for the city; and any area less than half, with such part of twice this supply as said area is of the entire area of the city; and a supply for at least five or six fire streams should be provided for the smallest area. *AG* should therefore carry $\frac{7}{8}$ of 1400 gals. plus 3000 gals., or 4100 gals. per minute. *G*4 should be given a capacity of $\frac{1}{3}$ of 1400 plus $\frac{2}{3}$ of 3000, or 2470 gals. per minute. In designing sizes of mains, provision should be made for any extensions of the system during the next few years (see Art. 7). Where more than one line of mains larger than 6-inch feeds a given section, the combined capacity of these may be made equivalent to the capacity computed as above, each carrying a

part of this proportional to the area directly fed by it, or to the importance of such area from a fire risk point of view.

A 6-inch line should not extend for more than 600 to 1000 feet between principal mains; nor a 4-inch for more than 400 or 500 feet. If the vertical streets in the figure are to be filled with 6-inch pipes, it is hence necessary to supply more mains crossing the 6-inch lines, as at TV, $Z3$, and 1–2. Each of these may be made of one-half the capacity of all the branches leading from it; for instance, TV, $Z3$, etc., may have a diameter of $\sqrt{\frac{3\times 6^2}{2}}=7.4$, 8-inch being the next largest commercial size. The remaining lines may now be made 4-inch, if this size be used; otherwise these also would be 6-inch. If the pipes are subject to tuberculation, 4-inch pipe should certainly be avoided, the effect of tuberculation upon the capacity being so great.

In the above the term " capacity " of pipes has been used as a convenient one, but " capacity with a given uniform friction head " is implied. If 4500 gals. per minute is to pass through MG, the loss of head and hence the pressure head at G can be found. If we fix a minimum pressure head of, say, 150 feet to exist at 2 with six fire streams playing in the section WQ, the head at G minus 150 will give that available for friction loss in carrying 1500 gals. of water per minute through $G4$ and 4–2. If we fix the capacity of 4–3 as $1\frac{1}{2}$ times, and $G4$ as $4\frac{1}{2}$ times, that of 3–2, we may *assume* a size for 3–2, as 8-inch, and 4–3 will then be $\sqrt{\frac{8^2(1.5)}{\sqrt{(1.5)^{\frac{6}{11}}}}}$ or 9.26 inches; and $G4$ will be $\sqrt{\frac{8^2(4.5)}{\sqrt{(4.5)^{\frac{6}{11}}}}}$, or 13.9 inches diameter. The head lost in passing 1500 gals. per minute through pipes of the given lengths and the above sizes is then calculated. If the total friction head thus found differs by more than 10 per cent from that available (less than 10 per cent need not be considered, as the commercial sizes of pipe will not permit of small changes in size), the assumed sizes—8, 9.26 and 13.9 inches—may be changed by

use of the formula $\left(\frac{f}{f'}\right)^3 = \left(\frac{d'}{d}\right)^{14}$, in which f and d are the total lost head as calculated and the assumed diameter of the pipe; f' is the available friction head, and d' the diameter sought. No considerable refinement in these calculations is called for, since the commercial sizes of pipe will be used; and the assumption that all the flow to 2 passes through 3–2 is by no means true, since probably a fourth of it or more comes through $G12$; also the estimate of maximum rate of consumption can be made by no means accurately.

Where it is supposed that an extension will at some future time be made, a special should be inserted for this, with a plug leaded and calked in the bell of the opening. At Q, for instance, a cross should be placed, and the right-hand branch plugged.

Art. 81. Standpipes and Tanks

Standpipes are made of wrought iron or steel, or of concrete; the majority in recent years being of steel, although good wrought iron is probably more reliable for this purpose. But good wrought iron has for years been difficult to obtain. A few special designs have been employed, but generally a standpipe is circular in plan and of uniform diameter throughout, resting upon a masonry foundation. It is sometimes, but not generally, roofed over. Standpipes are much exposed to winds, and hence should generally be anchored in some way, or provided with guys. When placed upon level ground, the lower 30 to 100 feet are in most cases useless for either pressure or storage, and this part of the pipe is frequently replaced by a tower, the whole thus becoming an elevated tank upon a tower.

The minimum height required for pressure is calculated by the methods already given. If the country be flat, to obtain a head which would enable fire streams to be thrown to the desired elevation by gravity might require a standpipe 200 feet or more high. Hence the domestic supply alone is frequently considered, and steam fire engines are relied upon for fire streams, the head being sufficient to bring to any hydrants and connected steamers the desired quantity. For this pur-

pose there should be some pressure at the hydrants when the mains are delivering the maximum fire requirement, allowance being made for friction in the mains. There should be contained in the tank, above the elevation required for this pressure, sufficient water to provide fire service while the boiler fires are being started up and the pumps enabled to supply the demand, even after a night's domestic consumption without pumping. This probably should be about one-fourth the daily maximum consumption plus the water required for fire service during thirty minutes. This storage is supplied by a combination of height and width. If the height be great, the pressure on the lower plates of the tank is excessive, as is that in the distribution pipes and plumbing, and the pumping must be against this great head. On the other hand, if the standpipe be made of great diameter both its cost and that of the foundation are increased.

STEEL STANDPIPES

Whether or no a standpipe shall continue to the surface or shall rest upon a tower is determined largely by financial and æsthetic principles. The difference in cost between tower and standpipe will not often vary greatly but for considerable heights or capacities the tower is probably the cheaper. A standpipe is at best an unsightly object, although some architectural beauty has been obtained by surrounding the pipe with a masonry tower in a few instances. A more graceful structure can be formed by a carefully designed steel tower, an example of which is given in Fig. 107, the elevated tank of the Iowa State Agricultural College, designed by Prof. A. Marston. As he states: "The only legitimate means for enhancing the architectural appearance of an engineering structure of this kind are to select pleasing proportions and graceful outlines, and to employ only neat, strong-looking details. Any use of sham ornaments is entirely out of place." This tank is 24 feet in diameter and 40 feet high, with hemispherical bottom, the total height being 168 feet. A few tanks have been raised upon masonry towers, but this construction is not common.

If a standpipe rather than a tank is used, the stability against overturning must be considered, and this stability increases directly as the diameter. The standpipe may be empty during a strong wind, and the resistance to overturning then depends upon the weight of the steel and the anchorage of this to the foundation. Calculations of several diameters and corresponding heights should be made to determine the most economical dimensions; each plan providing the desired water capacity above a certain elevation above the ground.

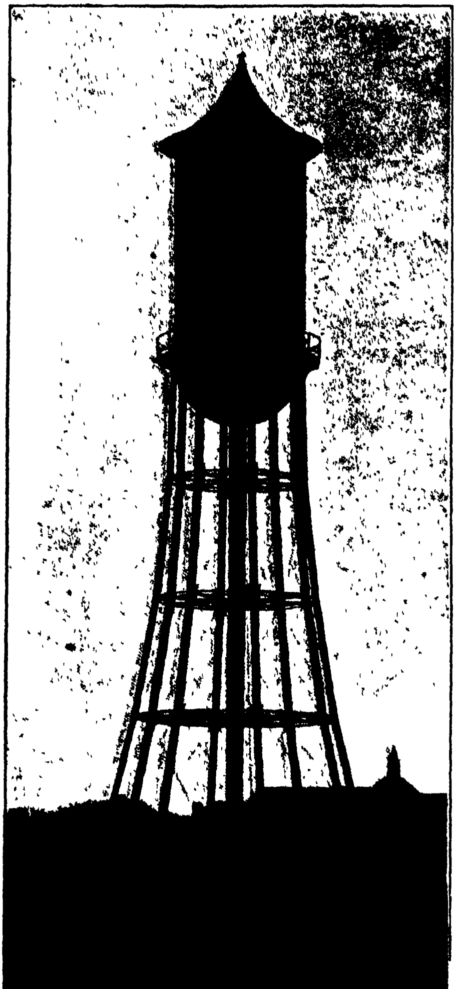


FIG. 107.—Elevated Water Tank.

The size of the tank having been decided upon, and its general design, the thickness of plates can be calculated. The stress in each side of the tank per vertical inch, due to the contained water, as in the case of water mains, is determined by the formula $S = \frac{.434Hd}{2}$, in which S is the tension in each side per vertical inch, H is the height of water in feet above the point in question and d is the diameter of the pipe in inches.

This stress must be resisted by the tensile strength of the steel,

including the vertical joints between plates. There is little variation in the stress except the slow change due to the rising or falling of water in the standpipe (that is, nothing resembling water hammer); the only additional stress that ordinarily can possibly be exerted horizontally is that due to ice freezing either on the surface of the water or around the inside of the shell. One or two standpipes have been ruptured by the expansion of ice so forming, but this is extremely unusual and will not happen with ordinary precautions. Consequently although the damage done by a bursting standpipe or tank may be considerable, a factor of safety of 4 is generally considered ample, or 5 if abundant safety is desired or there is danger from ice.

The plates and the horizontal joints between them have to sustain the weight of the plates above, but this weight is much less than can be sustained by single-riveted joints and plates of a thickness required by the hydrostatic pressure. Wind pressure tends to cause a tension on the windward side, thus lessening and possibly even exceeding the compressive stress; but it increases the compression on the leeward side. The wind pressure on a cylindrical surface is generally taken at 20 to 30 pounds per square foot of vertical section of the cylinder; and the pressure per lineal inch of seam on a leeward seam due to wind would not exceed $\frac{PH^2}{6\pi D}$; or, if $P=25$ pounds, $1.33\frac{H^2}{D}$; P being the wind pressure per square foot, H the height of tank above the seam and D the diameter, both in feet. The compressive stress per lineal inch of joint due to weight is $\frac{W}{12\pi D}$, or $.026\frac{W}{D}$, in which W is the total weight of that part of tank above the joint in question. Therefore the maximum total compressive stress on a horizontal joint is $\frac{2PH^2+W}{12\pi D}=\frac{1.33H^2+.026W}{D}$.

A standpipe is made of plates, generally 5 feet high between horizontal joints and 6 to 9 feet long measured around the circumference. The joints may be lap or butt (with the plates lapping a few inches, or with the edges brought into contact), the latter having a cover plate or welt on one or both sides of the joint

to hold the plates together, rivets being used in either case to bind the plates together. Punching rivet holes in a plate weakens it in the line of the holes. The tension on the joint is resisted by the shearing strength of the rivets and the friction between the plates at the lap caused by the binding effect of the rivets. The shearing strength of rivets is somewhat greater when they pass through three plates, the outer two of which pull in one direction and the middle one in the opposite direction; hence a butt joint with two cover plates (" double-welt butt joint ") is stronger than one with one plate or than a lap joint. It is more expensive, however, requiring more metal in both cover plates and rivets and more labor of riveters. The strength of a joint varies with the spacing and size of rivets; but in ordinary practice a lap or single-welt joint has the following efficiencies (ratio of joint-strength to strength of plate): Single-riveted joint, .54 to .58; double-riveted, .67 to .71; triple-riveted, .73 to .76. A double-welt triple-riveted joint should have an efficiency of .85 to .89. It is apparent that, by changing from a single-riveted lap joint to a double-welt triple-riveted butt joint, the plate could be reduced 36 per cent in thickness. This saving in metal would be partly balanced by the two cover plates and additional rivets, and the labor would be greater. Where heavy plates are used to resist high pressures, however, triple-riveted joints, either lap or butt, are commonly used.

The stress on a plate multiplied by the factor of safety and divided by the efficiency of the joint adopted gives the tensile strength necessary for the plate; and this divided by the strength of steel per square inch gives the thickness of steel necessary. The thicknesses of plates generally vary by sixteenths of an inch, and for each plate there is selected the next greatest standard thickness to that calculated for its lower edge.

For example, assume a tank 100 feet high, 25 feet diameter, triple-riveted lap joint, strength of steel 55,000 pounds per square inch. Then the thickness of the bottom plate would be $\frac{.434Hdf}{2se}$, in which f is the factor of safety, s the tensile strength per square inch of the steel, and e the efficiency of the joint, or

$$t=\frac{.434\times 100\times 300\times 4}{2\times 55{,}000\times .75}=.631.$$

This is a little over $\frac{10}{16}$, and $\frac{11}{16}$-inch plate should be used. If a single-riveted lap joint were used, the thickness would be $\frac{14}{16}$-inch.

The formula would give the top ring of plates a thickness of $\frac{1}{24}$ inch; but this would be too thin to hold its shape and would be collapsed by a high wind, and the thinnest plate used is $\frac{1}{4}$ inch, or occasionally $\frac{3}{16}$, and this should be stiffened by riveting an angle iron (bent to the proper radius) around the top edge. This minimum thickness is continued down to the point where the calculation demands a greater one; and the joints are made single-riveted down to the point where this no longer gives the required strength.

The relation between thickness of plates and diameter of rivets that will give maximum strength has been determined by the Navy Department to be as follows:

Thickness of plate	Less than $\frac{3}{16}$	$\frac{3}{16}$ to $\frac{1}{4}$	$\frac{1}{4}$ to $\frac{5}{16}$	$\frac{5}{16}$ to $\frac{1}{2}$	$\frac{1}{2}$ to $\frac{3}{4}$	$\frac{3}{4}$ to 1
Diameter of rivet.	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1

The centers of rivets should be distant from the edge of the plate at least $1\frac{5}{8}$ times their diameter. In double and triple-riveting the distance from center to center of rows of rivets should be at least $2\frac{1}{4}$ diameters in lap joints and $2\frac{1}{2}$ diameters in butt joints.

In case the horizontal joints are lapped, the rings may be truly cylindrical and every alternate one fit inside the rings next above and below; or they may be slightly conical, the upper edge of each being outside the lower edge of the one above. If the standpipe rests directly upon a masonry foundation, the bottom or bedplate has little tension to resist, but may rust more or less; and as the side or shell is riveted to it, it must have strength to resist the tearing out of these rivets when the shell tends to expand under hydrostatic pressure. Therefore it is customarily made three-fourths the thickness of the bottom ring, but not less than $\frac{1}{2}$ inch thick, unless the bottom ring be less than this, when both are made the same thickness. In

fastening the bedplate to the shell, an angle bent to the shape of the tank is riveted to both, either on the inside of the tank, or on the outside, the bedplate being extended beyond the shell for this purpose. The bedplate is made of plates riveted together with lap joints, the joints being laid out either rectangular or radial.

The overturning tendency of the wind must be resisted by the weight of the empty tank aided by anchor rods. The overturning moment is the product of height and diameter of tank, times wind pressure, times half the height, or $\frac{PH^2D}{2}$, which equals $15H^2D$ if P be taken as 30 pounds. The stability moment is $\frac{WD}{2}$, in which W is the total weight of the tank in pounds; but as this assumes that the entire weight could rest on one point in the bottom circumference without crushing, it should be reduced to $\frac{WD}{4}$ for safety. Then $15H^2D - .25WD$ is the moment that must be resisted by the anchor bolts. These bolts are spaced uniformly around the circumference, being anchored in the foundation and passing through brackets riveted to the outside of the bottom ring of the shell. Nuts on the upper ends of the rods are screwed down tight to the tops of the brackets. The sums of the moments found by multiplying the safe tensile strength of each rod (say 10,000 pounds per square inch) by the distance of each from a given chord at a distance $\frac{R}{2}$ from the center (the same chord being used for all moments) will give the resisting moment of the anchor rods. Or, calculating in another way; assume that each rod sustains the joint tension for a length of joint equal to $\frac{\pi D}{N}$, where N is the number of bolts. Then the rod tension is that per lineal inch of joint due to wind, less that due to $W\left(\frac{1.33H^2 - .026W}{D}\right)$, times one Nth the inches in the circumference of the tank. The foundation to which the rods are tied must have sufficient

weight to prevent being raised by the wind pressure, and the rods be so anchored in it by large plate washers as not to pull out of the foundation.

A manhole is usually provided at the bottom of a high tank to furnish entrance for cleaning, repairing and painting. This is a heavy casting riveted to the bottom ring and provided with a cast-iron cover that is fastened on by stud bolts and nuts.

A ladder should be fastened to the outside of the tank, generally beginning about 10 feet above the ground. In some cases a ladder is placed inside also in place of using a manhole, but this is bad practice in any but Southern countries, as the inside ladder is apt to be torn out by ice as water in the tank rises or falls.

A roof over a tank is desirable, to prevent heating the water, the growth of algæ, and pollution by birds and wind-borne dirt. Also a roof will often delay freezing. One objection, however, is that a cylindrical shell of ice freezing inside the tank and floating in the water may carry away the roof if the water in the tank rises considerably. For this reason it should be attached to the tank rather feebly, so that if this happens no other parts of the tank will be damaged or strained.

An overflow pipe is sometimes provided, generally carried down the outside of the standpipe and connected to the shell a foot or two from the top, to prevent the standpipe overflowing. This pipe is generally of wrought iron or steel and 4 to 6 inches diameter. Some tanks are provided with automatic alarms, by which the rising of the water above a certain elevation in the tank rings a bell in the pumping station. A tell-tale, giving in the pumping station continuous information as to the height of water in the standpipe, is desirable.

Usually one water pipe serves for both inlet and outlet. It should be provided with a valve just outside the standpipe. It is sometimes connected to the bottom ring of the shell, but a better plan, in the author's opinion, is to carry it through the foundation and connect it to the bedplate. This is especially desirable in Northern latitudes. The connection of this pipe with the bedplate is generally made by riveting to the bottom

a flanged bell casting, through which the pipe is passed, and in the bell an ordinary calked lead joint is made.

The foundation of the standpipe should be absolutely solid, generally of concrete on rock or other unyielding sub-foundation. The tunnel admitting the pipe to the standpipe should be substantially arched over. The top of the foundation should be perfectly level. Either a dry mixture of 1 part Portland cement to 1 or 2 of sand is spread about $1\frac{1}{2}$ inches thick over the foundation, or a layer of wet mortar equally thick, before the bottom is lowered onto it. The former is preferable, as it permits slight adjustment of the bedplate after the tank is filled, any slight leakage in such plate furnishing the water for setting the

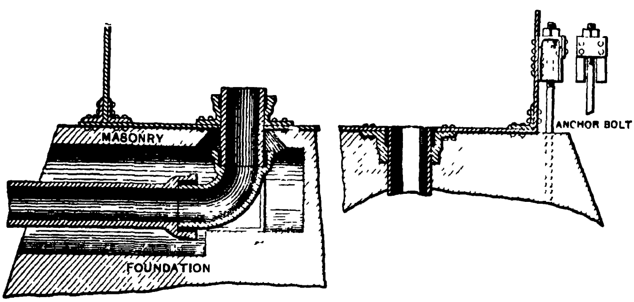


FIG. 108.—Standpipe Bottom.

cement. In any case the sand offers a cushion for the bottom and prevents all the weight from coming on the rivet-heads. Another method is to support the bottom about an inch above the foundation, fill around the edge with mortar, and when this has set, force grout under the bedplate through several holes in such plate.

"Tank steel" should never be used in standpipes. "Shell steel," the next best grade, is not advised for use. "Flange steel" is well adapted for standpipe use, being ductile and uniform in quality. It costs about 15 to 20 per cent more than tank steel, and 4 to 10 per cent more than shell steel. A good standpipe steel might be specified as "homogeneous steel made by the open-hearth process, having a tensile strength of 55,000 to 62,000 pounds per square inch; an elastic limit

of not less than 32,000 pounds per square inch; an elongation in 8 inches of 20 per cent in plates $\frac{3}{8}$ inch thick and under; of 22 per cent in plates $\frac{3}{8}$ to $\frac{3}{4}$ inch, and 25 per cent for plates $\frac{3}{4}$ inch thick and over, with a reduction of area of 40, 44, and 50 per cent respectively. A cold-bending test to be made without signs of distress as follows: for plates up to $\frac{1}{2}$-inch thick, flat on itself; for plates thicker than $\frac{1}{2}$ inch, 180° around a mandrel having a diameter $1\frac{1}{2}$ times the thickness of the plate. The metal to contain not more than 0.04 per cent phosphorus, nor more than 0.03 per cent sulphur." As but a few kegs of rivets are used in a standpipe, it is not the general practice to test the material for these at the works, but to obtain the best from reputable manufacturers. A field test of value is to cut the head from a rivet which has been headed in the work. If the head snaps off, the metal is brittle and unfit; but it should gradually cut and tear off.

The outside of the bedplate should be painted before it is lowered onto the foundation. After the plates are all riveted up, the inside should be well cleaned and given two or three coats of paint similar to that used for steel pipes. The outside should not be painted until after the tank has been filled with water, when all leaks should be stopped by calking. As soon as the tank is tight the outside should be painted. Slate or stone-color seems to be the least conspicuous color for large standpipes.

A standpipe is sometimes encased in a masonry structure, for the sake of appearance (a standpipe is at best a homely structure) and to protect it against freezing. Either stone, brick, or concrete may be used, and it should be designed by an architect to secure the best results. Such structures are generally provided with roofs. They of course add considerably to the cost.

ELEVATED TANKS

Elevated tanks are designed in the same way as standpipes, except for the bottom. This may be made flat and supported on a floor of steel beams, but most of those built in recent years have had hemispherical bottoms. The bottom is riveted to the

cylindrical shell, and the joint between the two sustains a stress per lineal inch equal to the weight of the tank-full of water plus that of the metal in the bottom, divided by the circumference of the tank in inches. The radial joints in the hemispherical bottom receive the stress $\frac{pr}{2}$, in which p is the normal hydrostatic pressure per square inch at the point in question and r is the radius of the sphere (which is also that of the tank). The stress on a circumferential joint is $\frac{W \sec \theta}{2\pi b}$, in which W is the total weight of water above the horizontal plane passing through the joint, θ is the angle between the vertical spherical radius and that passing through the joint, and b is the radius of the joint in the plane of the joint, in inches, $2\pi b$ being therefore the length of the joint. The thickness of plate, joint riveting, etc., are calculated as for the cylindrical plates; but the plates should be made about $\frac{1}{16}$ inch thicker to allow for stretching during the shaping. Tanks with hemispherical bottoms are made up to 30 feet or more diameter and 35 or 40 feet high. Generally the height is made 15 to 25 per cent greater than the diameter.

Stiffness and strength are best served by using a continuous curved or ring girder at the top of the tower that supports the tank, to distribute the thrust of the posts over the entire length of the bottom plate of the tank cylinder. For this purpose the lowest plate of the cylinder, made a little heavier then theoretically necessary, is used as the web of the girder, stiffened by vertical angle irons riveted to it; and angles are riveted to the top and bottom to serve as flanges. If N is the number of posts, r is the radius of the tank in inches, and W the total weight of tank and water, the maximum shear is $\frac{W}{2N}$; the maximum bending moment, in inch-pounds, is approximately $\frac{.53Wr}{N^2}$ (this is not exact, but is within 2 or 3 per cent for all cases); and the maximum torsional moment in inch-pounds is approximately $\frac{.32Wr}{N^3}$.

The tower must support the load, and also the effect of wind on tank and posts. The legs may be made straight, but a better effect is produced if they flare at the bottom, as in the Ames tower. As the stresses are similar to those in any steel tower, they will not be discussed. The number of posts may be between three and twelve, but either five, six or eight is economically and generally satisfactory.

The pipe to the tank is generally carried vertically up the axis of the tower. It should have an expansion joint near the tank. In cold climates it should be encased in wood or light steel to prevent freezing. It is generally supported in vertical line by surrounding it, at the level of each tower panel point, with a collar to which the horizontal diagonal tie rods are connected.

CONCRETE STANDPIPES

At least 25 standpipes and three elevated tanks have been constructed of reinforced concrete. In these, the reinforcing is relied upon to furnish the tensile strength, the concrete supporting the pressure between reinforcing rods and furnishing the impervious surface. When the tension on the steel exceeds 4000 to 5000 pounds per square inch, it will stretch so much that the concrete will begin to crack unless it be of sufficient strength to resist the rest of the tension. The reinforcement is generally made of large round bars, so that the concrete can be fully worked into all spaces. Chiefly because of this yielding of the concrete, it is believed, all concrete standpipes with two or three exceptions have leaked more or less; but this leakage has not been serious, the chief objection being the appearance caused, and many of them stop up in time. In several if not all cases where bids have been received for both concrete and steel standpipes, those for the former have been lower. A concrete standpipe does not rust nor need painting frequently, as does a steel one, and can be made more ornamental (or less of an eyesore).

As the shell expands when filled with water and the bottom does not, it is customary to tie the two together with heavy

reinforcement to prevent a crack forming at the joint. A more logical plan would seem to be to increase the annular reinforcement at the bottom of the shell to prevent expansion, or to provide a sliding joint between bottom and shell. The latter was used successfully in one case.

The shell is cast between sectional forms of steel or wood,

FIG. 109.—Erecting Concrete Standpipe.

Round reinforcing rods are seen at the top, the steel form plates in place below and at the left, with plates ready to be placed resting on the scaffold. Below the ring of form plates the concrete is seen.

which are moved up as the concrete sets, and never (so far as the author knows) in one complete form for the entire standpipe. Largely for this reason, the concrete is not carried up continuously, although each horizontal ring is made as a monolith. Leaks that develop generally occur in the horizontal joints between consecutive rings. To prevent this in the Westerly, R. I., standpipe, the top of each ring of concrete, after work was

stopped for the day, was scrubbed with brushes and water to remove the laitance and clean the surfaces of all exposed stones and at the same time the strips of wood were removed which had been embedded in the concrete to form bonding grooves in the joint; and before concrete was placed the following day, the joint surface was thoroughly washed and coated with neat cement rubbed in with a brush. Another plan is to use long copper strips 4 to 6 inches wide with a small crimp in the middle, placed as a ring around and in the middle of the wall, half the width being imbedded in the concrete and the other half protruding to extend into the following ring, the ends of the copper strips lapping 3 to 6 inches. Flat steel strips 6 inches wide and $\frac{1}{8}$ to $\frac{1}{4}$ inch thick have been used in the same way.

Ordinarily the rings of steel reinforcement are calculated for a stress of between 12,000 and 16,000 pounds per square inch, the ends being lapped and so fastened with clips as to develop the full strength of the rod at the joint. These are generally supported in place by vertical angles or channels, which also serve for vertical reinforcement; although in one standpipe no vertical steel was used. The number of rings, or the spacing, or diameter of rods (or two or all) are varied to provide for the variation in hydrostatic pressure with the depth. Some claim the steel must be within 2 inches of the outer surface to be most effective. The top of the standpipe shell is made between 6 and 12 inches thick. The bottom of the shell has been made from 9 to 36 inches thick.

The concrete is generally made rich to secure water-tightness and strength; 1 : $1\frac{1}{2}$: 3, or even 1 : 1 : 2 being not uncommon. The aggregates should be selected to give maximum density, with abundance of the finer; enough water should be added to produce a quaking consistency and the concrete be lightly rammed or joggled to secure the escape of all air or free water and thorough filling around the reinforcing; the mixing should be especially thorough; the forms should be tight to prevent the washing out of cement with escaping water; they should be firm, and should be of metal to secure a surface skin, which is more impervious than several inches of concrete. After forms

have been removed from any surface, it should be kept wet until the concrete is at least two weeks old, and protected from the sun by placing over it burlap, planks, etc.

Leaks that develop are difficult to stop. Plastering with cement on either inside or out is useless. Bituminous water-

TABLE No. 48

REINFORCED CONCRETE STANDPIPES

Location.	Diam., Ft.	Height, Ft.	Cost.	Cost per 1000 Gals.	Remarks.
Brunswick, Me...	97	46			
Paris, Me.......	80	14	$7,150	$13.60	No roof.
Littleton, N. H ...	27	27	2,080	18.10	
Littleton, N. H ..	17	17	1,080	38.60	
Burlington, Vt...	30	30	5,050	31 40	On tower 25 feet high.
Attleboro, Mass...	50	106	36,000	24 50	Shell 18″ at bottom, 8″ at top.
Bridgewater, Mass	30	78	....	....	Never leaked at all.
Chelmsford, Mass .	45	16			
Fall River, Mass..	16	15	850	38.60	
Hull, Mass.......	20	50	....	....	Fort Revere, for army; first ever built.
Leicester, Mass. ..	80	30	12,380	11 00	No roof.
Lexington, Mass..	30	104			
Manchester, Mass	60	72	36,000	23 60	
Medfield, Mass....	20	23	1,530	27 80	
Medfield, Mass....	20	20	1,400	29.20	
Middleboro, Mass..	41	173			
Rockland, Mass...	46	104	....	....	Shell 36″ at bottom.
Waltham, Mass...	100	43	25,785	12.90	Concrete roof on steel trusses; shell 18″ at bottom, 12″ at top.
W. Brookfield, Mass	50	20			
W. Falmouth, Mass.	40	40			
Fulton, N. Y.....	40	100			
Jamestown, R. I...	35	50			
Warren, R. I.	40	50	10,590	22.50	
Westerly, R. I....	40	70	17,960	24.30	Shell 14″ thick throughout.*
Woonsocket, R. I..	79	42			
Danbury, Conn...	20	30	7,250	103.00	On tower 25 ft. high.
Madison, N. J....	25	130	16,000	33.40	Around old steel pipe 75 ft. high.
Milford, O........	14	81	....	....	Shell 9 in. thick.
New Ulm, Mich...	75	30	....	....	Shell 20″ at bot., 15″ at top.
Anaheim, Cal.....	32	38	11,400	....	On concrete tower 70 ft. high. Shell 5″ at bot., 3″ at top.
Kitchener, Ont....	50	76			

* Cost of concrete: labor, $1.70 per cu. yd.; plant and staging, $2 14; cement and limoid, $3.34; aggregate, $1 56; total, $8.74. Reinforcement and labor, $7.73 per ton; staging, $4.50 per ton, material, $38; total, $51.23 per ton. Finish, $1.17 per 100 sq. ft.

proofing surface applications on the inside have been successful in some cases. In Waltham a small leak was stopped by calking in lead wool. At Attleboro, with the tank full, at each leaky spot a hole was cut in to the reinforcing, made wider inside than on the face, and filled with small crushed stone. This was covered with a patch of mortar and wire netting, and, when this had set, mortar was forced in under pressure furnished by a tank of carbonic acid gas.

Table No. 48 is believed to be a practically complete list of concrete standpipes built to date. The first was at Fort Revere, Hull, Mass., built in 1902 for the U. S. Army.

ART. 82. ESTIMATING COST

It is always desirable, and frequently required by law, that a careful estimate be made of the cost of any proposed public work. For this purpose in connection with water works, map, plans, and specifications should be carefully studied to obtain quantities; the character of soil or rock to be excavated should be ascertained, the cost of getting supplies to dams and other parts of the system considered, and in general as careful a study made of the conditions as a contractor would make before bidding. Also the prices of material and supplies should be obtained, including the cost of getting them upon the ground. From these should be made as close an estimate as possible of the actual cost of constructing the system.

The cost of any piece of work consists of the cost of the labor and materials of all kinds entering into it; an allowance for plant sufficient to pay interest, repairs and depreciation or obsolescence on all tools and appliances used—picks, shovels, wheelbarrows, engines, etc.—and the power for operating, whether horse, gasoline or steam; cost of foremen and all other supervision, timekeeper, office expenses, labor insurance or an allowance in lieu thereof to cover damages recoverable by injured workmen; interest on the money put into the work up to the time payment is made for it; demurrage, temporary roads, and possibly other items also; and finally a fair profit.

Of these, rate of interest on plant cost and other money advanced and labor insurance are fairly constant; some materials, like iron pipe, lead, etc., change in price slowly in ordinary times and can be calculated very closely for a given piece of work; while other materials such as clay, rock, sand, and other local products vary several hundred per cent in different localities. Wages change slowly, but vary considerably in different sections, and the amount of actual work that is obtained for a dollar varies with the character of local labor, the ability of the foreman to "get work out of the men," and of the contractor to plan the work to the best advantage.

It must be borne in mind that prices of labor and of materials vary. If the prices of iron and copper go up, then so will those of iron pipe, valves, fire hydrants, meters, etc. The engineer or contractor must learn for each job the costs of labor and materials that will prevail at that time and place.

On simple pick and shovel work, labor, foreman, and timekeeper, plus a charge per man per day for the pick and shovel used, may cover all the expenses. But if a steam shovel, dinkey engine and train, steam roller, and power-driven concrete mixers are used in a reservoir construction job, in which floods may interrupt the work and even cause some losses, the calculation of probable cost is not so simple. An amount per day should be allowed for each piece of apparatus sufficient to pay interest on its purchase cost and repay such cost during the life of the apparatus. Suppose, for instance, that a steam shovel costs \$6000, will last five years with a salvage value of \$1000 at the end of that time, and can be used an average of 150 days a year; and that money can be borrowed at 5 per cent. Then there should be charged against the shovel each year $\frac{\$6000-\$1000}{5}+(\$6000\times.05)$ $=\$1300$, or \$8.66 for each day used; and if it shovels 900 cubic yards a day, there should be a charge against each cubic yard of .96 cent for steam shovel interest and depreciation. Similarly all other plant should be allowed for.

In addition to this, there are general charges to be allowed for in the cost, such as cost of moving the plant to the work

and from it again, storing it during the winter, keeping it painted and repaired, etc.; making temporary roads for and bringing in material (it may cost $3 for hauling to an inaccessible spot $1 worth of pipe or other material); diverting streams around dam sites; barns, office and other buildings; bringing water to the work; and possibly other expenses, all of which must be distributed over the several items in making itemized estimates or bids. It follows, of course, that the more extensive the work the less the amount of these general charges per unit of work—that is, the less the unit cost.

Many of these general costs must be estimated for each job, others have a more or less constant value. For instance, allow about ¾ cent a day for each pick (this does not include sharpening, cost of which will vary with nature of material picked and number of picks to be sharpened daily); ½ cent for each shovel; 5 cents for a wheelbarrow; $15 to $25 per day for a steam shovel, depending upon size (this includes engineer and helper, and fuel, as well as fixed charges); 10-ton steam roller, $10 to $15; dump wagons, 20 to 30 cents; drag scrapers, 5 cents; wheel scrapers, 10 cents. These are offered as approximate for average use, and assuming they are kept in good condition. It is assumed that they will be in actual use about 125 to 150 days a year.

In the following pages are given costs of labor and material based on average conditions. To these must be added plant costs and other overhead and general charges. *The labor costs are based on 15 cents per hour for common labor and 35 cents for two horses and driver. The prices of materials are based on those prior to the war.* At present (1924) they are 50 to 200 per cent greater than this, and if used in making estimates these costs should be increased in proportion.

Clearing and grubbing reservoir sites. The cost of this may often be more than offset by sale of timber cut for lumber, firewood, or other purposes. The actual cost of clearing ordinary timbered land, with labor at $1.50, has generally been between $25 and $200 per acre, depending upon the size, number and kinds of trees, amount of underbrush, irregularity of sur-

face, etc. If the land is very swampy, is covered with scrub second-growth, or if large numbers of boulders must be removed, the cost may exceed $200.

Excavation of loam or other easily pulverized soil by drag scraper, including first loosening it by plow, can be done for about 1½ cents per cubic yard for loosening by plow, and 5 cents for handling by scraper, plus 4½ cents for each hundred feet of haul in excess of 100 feet. Add 1 cent for foreman and wear on scraper. Total 7½ cents plus overhaul. For stiff clay add 25 to 50 per cent to the first two items.

If wheel scrapers are used, deduct about ½ cent from the second item, and allow for overhaul 3 cents to 1½ cents, depending on size of scraper. Loosening by pick costs from 1 cent in easy earth to 11 cents in very stiff clay or hardpan; about 4 cents for average earth. Picking and loading into wheelbarrows, and dumping will cost about 17 cents a cubic yard for easy digging, up to 40 cents for hardpan, and wheeling 5 cents per 100 feet for steep runs and 3½ cents on the level.

Loading into carts or wagons will cost about 12 to 40 cents, say 15 cents for average earth. Hauling by one-horse cart costs about 4 cents plus ¾ of a cent per 100 feet of haul. Hauling by two-horse wagons costs about 6 cents plus ½ of a cent per 100 feet of haul. Excavating by steam shovel costs about 6 to 10 cents depending on the percentage of its full capacity that can be handled and nature of soil; the cost may run very much higher if dirt cannot be removed rapidly. Handling by wagons will cost about 1 cent less if excavation is by steam shovel than if it is by hand shoveling, less loading time being required.

Spreading earth in 6-inch layers on an embankment costs about 2 cents a cubic yard if the man is kept busy, more if he is not.

Rolling thoroughly with a horse-drawn roller costs 1 to 2 cents per cubic yard. Add cost of sprinkling, which will be influenced by accessibility of water. Dressing slopes may cost from 3 to 30 cents per square yard, depending upon skill of laborers.

Lining banks with dry stone paving 12 inches thick costs 20 to 40 cents per square yard, plus cost of stone and of gravel or sand bedding. If the stones are irregular in shape and need

much dressing this increases the cost. Concrete lining 6 inches thick costs about \$1.50 to \$1.75 per square yard on slopes, \$1.25 to \$1.50 on a reservoir bottom.

Puddle in a trench, rammed and trodden into place, costs 40 to 60 cents a cubic yard in addition to materials delivered on the spot. Puddle made in wide layers and rolled with a heavy grooved roller costs 20 to 40 cents.

Stone masonry for dam faces, coursed ashlar, costs \$18 to \$25 per cubic yard, varying with the hardness and cost of stone. Dimension stone for gatehouses, etc., may cost double this. The largest part of the cost of ashlar is in the dressing. Rubble masonry costs \$4 to \$8.

Timber in foundations will probably cost \$5 to \$20 per M. B. M. for labor, above the cost of material, the cost depending upon the amount of framing involved.

Concrete. The cost of this is a combination of the cost of the sand, stone or gravel, and cement of which it is composed; the labor of mixing it, including cost of coal or other power used; cost of placing it; cost of placing and removing forms; and depreciation on all the plant used. According to figures collected by "Municipal Journal" of the cost in 1916 in nearly a thousand municipalities of sand and broken stone or gravel, sand varied from \$0.20 to \$3 per cubic yard, averaging \$1; broken stone varied from \$0.40 to \$3.75 per cubic yard, averaging \$1. Cement cost about \$1.25 per barrel. Labor feeding mixer by wheelbarrow, mixing, and delivering direct from mixer by chute or beam and bucket, \$3 to \$4 per cubic yard. When mixed in large quantities and mixer fed direct from bins, this last cost may be reduced by 25 to 50 per cent. For slope lining and other construction requiring special care in laying, add 25 to 50 cents a cubic yard. The cost of forms may be as little as 10 cents a cubic yard or as much as \$5.00, depending on the elaborateness of the forms and the relative volume of concrete placed therein.

Cast-iron pipe, in carload lots, per ton of 2000 pounds, has cost as stated in the accompanying table for the past seventeen years. This is the cost of pipe larger than 4-inch. For 4-inch and smaller, the price in 1916 was \$3 per ton greater.

Table No. 49

COST OF CAST-IRON PIPE, 1900 TO 1916

F.o.b. New York, short tons, carload lots

	1900	1901	1902	1903	1904	1905	1906	1907
January....	$27 50	$21.75	$24 50	$29.25	$24 50	$28.00	$29.75	$34.25
February...	26.75	22.25	25.00	29.25	24 25	28.50	29 50	34.25
March....	26.75	21.50	26.25	30.75	24.25	26.75	30 50	34 00
April.....	26 50	22 00	26 00	31 00	24.25	27 00	29 75	33.50
May.......	26.00	22.25	27 75	30.75	24 00	27.25	31.00	34.25
June..	24.50	23 00	28.00	35 75	23 50	27 25	32 50	33 50
July	24 75	23.75	28 50	30 75	23 50	27.25	30 25	34.00
August.....	23.50	23 75	29 50	29 50	23.50	27 75	30 50	32 50
September..	22.25	23 50	29.50	29 00	23 00	27 25	31.00	33 00
October...	21 75	24 00	29 50	26 00	23 25	28 25	33 00	33 50
November	21.75	24.50	30 75	24 50	25 00	29 00	33.25	28 50
December	21.75	23.75	29 25	24 25	27 00	29 25	35 50	28.00

	1908	1909	1910	1911	1912	1913	1914	1915	1916
January...	$27 00	$24.50	$25 50	$22.00	$22 00	$25 00	$22 00	$20.00	$29 00
February.	26 75	24 25	25 50	21.50	22 00	24 75	22 00	20.00	29 33
March	26.25	25 25	25.50	21.00	22.00	23 87	22 00	20 00	29 75
April.	26 25	25 00	25 50	21 00	21 25	23 50	22.00	21 60	30.50
May.......	26 25	25 25	25.50	21 00	21.00	23 00	20 88	22.00	30.50
June......	25 75	26.00	25 25	21 00	21 00	23 00	20 50	22.25	30.50
July.......	25.75	26.25	24 00	21.00	22 10	23.00	20 50	22.50	30.50
August. ...	25 25	26.00	23 50	21.00	22 00	23 00	20.50	23 25	30 50
September..	25 75	25.75	23.50	21 00	23 12	23 00	20.40	24.37	30 83
October. ..	25 75	25 50	23 00	21 00	24.50	23 00	20 00	25 25	31.50
November.	25.00	25.87	22.12	21.40	24 12	23 00	20 00	26.50	35 50
December	25.50	25 70	22 00	22 00	24 62	22 33	20 00	27 60	41.00

The cost per foot of the various sizes and classes of pipe (Am. Water Works Association Standard) can be found by multiplying the following units by the cost per ton in dollars.

COST OF CAST-IRON PIPE PER LINEAL FOOT AT $1 A TON

Diameter of Pipe, Inches.	Class *A*, 100-ft. Head.	Class *B*, 200-ft. Head.	Class *C*, 300-ft. Head.	Class *D*, 400-ft. Head.
4	.0100	.0109	.0117	.0125
6	.0154	.0167	.0179	.0192
8	.0215	.0238	.0260	.0279
10	.0286	.0319	.0354	.0384
12	.0363	.0411	.0459	.0500
16	.0542	.0625	.0719	.0792
20	.0750	.0875	.1042	.1146
24	.1021	.1167	.1396	.1534
30	.1459	.1667	.2000	.2250
36	.1959	.2271	.2729	.3125
48	.3334	.3750	.4542	.5250

Trenching for pipe by hand, sheathing if necessary, and backfilling costs about as follows for easy digging—loam, sandy clay, etc. Sand and gravel are included as " easy," but must be sheathed. Hardpan will cost about double these figures. Rock excavated in the trench will cost from $2 to $10 per cubic yard—the latter when only 2 or 3 inches depth are to be removed and the rock does not shatter readily.

COST OF EXCAVATING AND BACKFILLING PIPE TRENCHES IN EASY MATERIAL; NO GROUND WATER, NO STREET PAVING. CENTS PER LINEAL FOOT

Depth of trench, feet.	4	5	6	8	10
4-inch to 10-inch pipe	8 3	9 8	11 3	15 0	21
15-inch pipe	9 8	12 0	13 5	18 7	26
20-inch pipe	11 3	13.5	15 7	22 5	32
24-inch pipe	12 7	15 7	18.0	26 5	37
30-inch pipe	15.0	18 0	21 0	30.0	42
36-inch pipe	17.2	20.2	23 5	33 0	46
48-inch pipe	21 0	24 2	28 0	38.0	53
Close sheathing	20 0	21 0	22 0	26 0	32
Sheathing, planks 4 ft. apart	9 0	10 0	10 0	12.0	15

COST OF LAYING CAST-IRON PIPE; CENTS PER LINEAL FOOT

Size of pipe.	4	6	8	10	12	18	24
Unloading, hauling and distributing.	3-1	5-2	7-3	1-4	1 5-5 5	3-8	4-12
Laying.	6	7	1 0	1 5	2 0	3 0	5 0
Lead, at 4 c a pound	2 9	4 1	5 2	6 3	7 5	13.0	17 5
Yarn, at 4 c. a pound	04	06	07	09	11	14	.2
Coke and furnace man	.2	2	25	30	36	70	1 0
Yarning and calking	1 0	1 4	1 8	2 5	3 0	5 0	7 5
Total	5 0-5 7	7 0-8 5	9 0-11 3	11 7-14 7	14 5-18 5	24 8-29 8	35.2-43 2

COST OF GATE VALVES, IRON BODIES, BRONZE MOUNTED, BELL ENDS. TESTED TO 300 POUNDS PRESSURE

Size of valve..	4 in	6 in.	8 in.	10 in.	12 in.	14 in.	16 in.	20 in.	24 in.
Cost	$8.00	$12.00	$18.00	$26.50	$37.00	$52.50	$67.00	$135	$220

Fire hydrants, 6-inch, 2 hose nozzles and a steamer nozzle, 5 feet from ground to branch, cost about $25 to $30; add about $3 for each independent nozzle gate.

Water cranes cost about $25 to $40, for 3-inch or 4-inch connections to main.

Steel riveted pipe 30 or more inches in diameter weighs from 3 to 8 times, in pounds per lineal foot, the diameter in inches; the former for a safe working pressure of 80 pounds per square inch, the latter for 150 pounds with the largest sizes and 250 with the 30-inch. The cost of pipe is 3 to 4 cents per pound. The cost laid under average conditions is about 4 to 6 cents per pound, or 15 to 33 cents per inch diameter, including excavation and all charges.

Lap-welded pipe weighs from $.67d^2$ and $.94d^2$ (for standard and extra strong, respectively) for 4-inch, to $.34d^2$ and $.45d^2$ for 12-inch, d being the diameter in inches.

Tube wells are generally sunk by contract, at a certain amount per foot, and the cost varies with the material, depth, and size of casing. Rock costs much more than sand or clay. Including casing, a 6-inch well will cost perhaps $3 per foot for the first 200 feet, and $4 to $5 below 700 feet; although where wells are common and there is competition the price may be cut to two-thirds this.

Standpipes of steel cost about 2.5 cents per pound of steel for erection, this covering labor, rivets and tools and painting. Add about 3.0 to 3.5 cents per pound for the steel plates. Additional for the foundation. Weight of steel is of course calculated from thicknesses obtained in designing the structure. Towers cost $1 to $1.50 times the product of the height of the tower and the mean of its top and bottom diameters.

Filtration Plants. The cost of slow sand filters, averaging local conditions, is about $20,000 to $35,000 per million gallons daily capacity; this including all costs except land and pumping machinery. Amount of grading required may considerably affect the cost. Cost of rapid sand filters averages $12,000 to $18,000 per million gallons daily capacity. The cost given for slow sand filters includes sedimentation basin or preliminary filters, and clear-water reservoir. That for rapid sand filters includes coagulating and clear-water basins.

Operation and maintenance of slow sand filters cost $2 to

\$4 per million gallons filtered; of rapid sand filters, \$5 to \$8 per million gallons, including chemicals.

The correct cost basis is the annual cost—that is, operating expenses plus interest and depreciation on construction cost. Depreciation probably should be taken as not less than 5 per cent nor more than 10 per cent.

The cost of an up-to-date pressure-filter plant of 1,000,000 gallons capacity is given by Harold C. Stevens (in 1916) as follows: Filters, including foundation, \$5000; controlling device, \$1300; settling basin, \$3500; low-lift pumps, \$1200; superstructure, \$1500; total \$12,500. For large plants this would be reduced to \$9000 per million gallons.

Pumping Engines. Steam engines vary in cost with type, size, head pumped against, steam pressure, and cost of iron and bronze or brass. The annual cost is the sum of interest and depreciation on construction cost of pumps and boiler plant, foundations and building, and the annual cost of coal, lubricants, maintenance, and repairs, and engine and boiler room force.

$$\text{Cost of coal per year} = \frac{990PTC}{DE},$$

in which P is average indicated horse-power developed;
T is number of hours pumping per year,
C is cost of coal in dollars per 2000 pounds;
D is duty expressed in foot-pounds per 1000 pounds of steam;
E is evaporation factor of boiler (about 8 pounds per pound of ordinary good bituminous coal).

The cost of a pumping engine complete, with foundation, piping and appurtenances, is about as follows, per million gallons capacity per twenty-four hours: Compound, condensing, low-duty, horizontal, \$2300. Low-duty, triple, condensing, horizontal, \$2800. Cross-compound, condensing, horizontal, \$3300. High-duty, triple, condensing, vertical, \$4800. These figures do not include building or chimney, nor boilers. Boilers

with mechanical stokers, feed pumps, appurtenances, and steam piping, ready for service, cost about \$15 to \$20 per boiler horse power. These prices are based on a head pumped against of about 200 feet.

The cost per million gallons capacity of a complete pumping station, using reciprocating engines, triple expansion, with high-pressure boiler, brick building—everything but land—is given by C. A. Hague as \$6000 plus \$25 $\times P$, P being the pressure pumped against in pounds per square inch. (All of these were prices in 1911).

CHAPTER XV

SUPERVISION AND MEASUREMENT OF WORK

ART. 83. RESERVOIRS

AFTER a reservoir site has been cleared, but before any excavation is begun, levels should be taken of the entire ground surface where there is to be either excavation or embankment, and a contour map of the site prepared, with 1-foot intervals between contours. After the completion of the reservoir, levels are again taken, referred to the same bench marks and base line, and the amount of excavation and embankment calculated by a comparison of these with the previous levels.

Before excavation, stakes for the direction of the contractor should be placed at frequent intervals along the lines of the inner and outer toes of the embankment, including in this the 1 or 2 feet on each side which is later to be removed in shaping and dressing the slopes. It is desirable to place monuments well beyond all danger of interference by the work, to which the center line of the embankment or dam is referred; they being on each end of an extension of the center line, if this be straight.

During the construction of embankments the points requiring attention are: The width; each layer must extend the full required distance beyond the finished face. Depth of layers; this must at no point be greater than specified. Watering; the layers should be sprinkled uniformly in all parts, when wetting is required by the character of the material. Character of material; each load should be inspected, and no large stones, roots, or other vegetable matter, or material different from that specified, should be permitted to be dumped. Form of surface; the edges should be kept higher than the center.

Rolling; the roller should be of the required kind and weight, and pass over each spot until it becomes solid and homogeneous.

If a masonry core wall is used, this should be kept above the embankment, and the latter should be thoroughly tamped where the roller cannot reach it. In many cases the core wall is practically completed before the embankment is begun.

Particular attention should be paid to the puddling or concrete around any pipes which pass through the embankment; and to the total removal of all top soil under the embankment, and the compacting of the first layer with the ground thus uncovered. To insure the last, the ground should be plowed over after the removal of the top soil.

After the completion of the embankment, templets are set and the slopes trimmed by these. On curves, templets should not be more than 25 to 50 feet apart, depending upon the radius. On tangents they may be 50 to 100 feet apart. Each templet is formed by driving a stout stake at the foot of the slope, and others 10 or 12 feet apart directly up the slope from this. Care should be taken that these stakes are in a plane normal to the axis of the embankment or to a tangent to its curve at this point. To these stakes planks are nailed in such a position that the finished slope will be a fixed distance, say 1 foot, under their lower edges. The slope is then carefully cut down to the required surface.

Masonry work requires the most careful inspection, and if masons new to this class of work are employed it is frequently necessary to discharge half of them before the remainder can be compelled to obey instructions. It must be remembered that the masonry must be not only strong, but also water-tight. The rock upon which it is to rest must be cleaned with brooms and washed, and should be covered with a thick bed of rich mortar just before the masonry is started upon it. Every stone used should be perfectly clean of dirt and dust. It should be *lowered* into its bed and not rolled or barred into place. Each bed should be covered with an excess of mortar, and no mortar should be "slushed" in after the stone is laid. It is absolutely essential that no spall be driven under a stone

after it is in place, to level it up or for any other purpose; but spalls and quarry-chips may be used in leveling a bed before the stone is lowered onto it. In using spalls and chips, however, these should never be placed in the bed and mortar spread over them, but the mortar should be spread first, and the spalls forced into it. All side and end joints as well as beds should be made in this way. There should be no grouting of dam masonry, for there should be no spaces for grout to penetrate. There should be no dressing of stones on the wall, but this should be done on the scaffold, or while the stone is suspended from the derrick. Courses should not be leveled up at all, but should be left as uneven as possible. The above are essential to a *tight* wall, although one on which less pains have been taken may be sufficiently *strong*.

The mortar should be rich—not more than 2 of sand to 1 of cement—and thoroughly mixed before wetting, until it is absolutely uniform in color throughout. The mixing is fully as important as the proportions of cement and sand. A slow-setting cement is preferable; and mortar should never be retempered for use, but on taking its initial set in mortar box or board should be at once thrown away. If the work is done by contract, it is a good plan for the party of the first part to supply the cement, and thus remove the chief incentive for parsimony in its use.

Concrete should be thoroughly mixed, and immediately put in place. The broken stone should be free of all dust or dirt. Too much water will cause the concrete to be honeycombed and porous; there should be such an amount that light ramming will just bring moisture to the surface. Hard-wood forms the best material for rammers; and the ramming should never be heavy, but just sufficient to compact the material. Special care should be taken to remove from concrete that has set all loose particles and laitance and wet it, before more concrete is placed on it. It is advisable to give the up-stream face of a concrete core wall two or three washes of cement and water to increase its imperviousness. All concrete should be kept damp until set, and should be shaded from the sun if possible.

Concrete in slope walls cannot be mixed very wet or it will slide down the slope when placed; nor can it be carried too far up the slope continuously, or the weight will force out that at the bottom unless it is made too dry to produce a water-tight lining. About 12 to 15 feet is generally the greatest height that can be laid at a time successfully unless the slope be flatter than $1\frac{1}{2}$ to 1. The consistency of the concrete should be such that after going over the entire surface three or four times with a rammer the concrete will just begin to quake. The concrete must be thoroughly compacted and mortar brought to the surface at all points, but it must not be soft enough to slide down the slope. A continuous strip 10 to 15 feet wide can be carried around the entire reservoir, but it is generally considered preferable to insert expansion joints about 20 feet apart. One advantage of this is that the lining can then be placed in alternate blocks, using forms at the edges as well as the top to insure the proper thickness and surface. The slabs may be screeded and a surface of cement mortar applied and troweled down to a smooth finish.

If the bottom of the reservoir is in rock the bottom lining can be made without expansion joints, the bond with the rock preventing movement. The rock should be scrubbed clean before placing the concrete, and if it be smooth it should be plastered with cement mortar before the concrete is placed. Unless expansion joints are used, an edge against which new concrete is to be placed should be cleaned of all loose material and laitance and thoroughly plastered with cement mortar just before the new concrete is placed. In all cases of either bottom or side, the edges of a block or strip under construction must be made a plane surface normal to the top surface of the concrete, by use of forms held solidly in place, and spaded to force all stones away from the form; and work, whenever stopped temporarily, must end up with such a face. A sloping edge should never be given to concrete, because this would cause one block to rise upon another if expansion takes place, and for other reasons also.

For expansion joints no material containing felt or other made joint should be used, but the joint should be poured with a tar or asphalt material of a consistency as soft as can be used without

running in hot weather. No expansion joint is needed between rings or tiers on the slope, which can be joined by a mortar bond, as the weight of the upper rings will maintain the contact.

For a distance of 12 to 20 feet above the ground a masonry wall can be laid to templets, carefully set and tested daily. Above this point the faces of the wall can be set by offsets from a transit line run through its center; or by plumbing up from known points in the face below, the transit line being occasionally used for a check; the length of offsets depending, of course, upon the elevation of the point considered, with reference to that of the crest, which elevation must consequently be known. A convenient method for obtaining these elevations on low dams is to so place a level that its H.I. is the elevation of the crest; the rod-reading then being always the exact distance below this of the point on which is it held.

Ordinarily the test of a dam or reservoir is filling it with water. This should be done slowly, particularly in the case of earth embankments, slight porosities in which will close slowly by absorption of water unless the head above them be first made so great as to cause rapid percolation and leaks. Both masonry and earth dams may leak slightly for a few days or weeks after filling, but ultimately become perfectly tight. If, however, a leak begins to increase in volume, or the water is turbid, the reservoir should be emptied at once and the leak found and stopped.

Earthwork is generally paid for by the cubic yard, and is measured as described above. Masonry also is paid for by the cubic yard, there being generally two classifications in dams, face ashlar and rubble backing, with a small amount of dimension stone in gatehouses and ornamental work. The face masonry may be paid for by the square foot of surface; or considered as extending for a given distance from the face into the wall, and paid for by the cubic yard. Gatehouses and similar structures may be contracted at a lump sum for all above the foundation; the latter being paid for by the cubic yard, since its depth is not generally known exactly beforehand.

Clearing and grubbing are generally done by the acre; the

removal of top soil by the square or cubic yard. The handling of the water of any streams which flow across the work may be made a lump-sum item of the contract, or may be considered as included in the construction items.

Lining, whether of concrete, puddle, slope-wall, or other material, is generally paid for by the square yard. The area is determined by a careful survey of the bottom, and of the upper edge of the slope lining, together with measurements from top to bottom of slope at 25 or 50-foot intervals.

ART. 84. CONDUITS AND DISTRIBUTION SYSTEMS

Before the construction of a bench flume, slope stakes should be set as for a railroad cut (except that no part of the flume should come upon the fill), and levels of the surface be taken. After grading, center stakes are set and the foundation piers or sills are constructed, care being taken to have these the proper distance apart and at the exact elevations desired. In the construction of the flume, the dimensions and grade of the flume should be made to correspond exactly to the plans.

The masonry in canal walls or lining should be built in the manner specified above for dams and reservoirs. Canal embankments also should be built as are earth dams.

In the case of stave pipe the staves should be examined for wanes, shakes, splits, knots, or other defects; and the bands and shoes, for flaws of any kind. An eye should be kept upon the cinching, to see that it is not made too tight; and upon the butt joints, to see that each is driven " home." A table should be prepared beforehand from the profile of the line, showing the proper spacing of bands at each station; and the stations should be designated upon the ground by stakes properly marked.

In constructing riveted pipe, care should be taken to prevent abrasion of the coating, and if this occur, the coating should be renewed by use of " P. & B." paint or other satisfactory compound. Rivet holes should line up within a sixteenth of an inch, and rivets be well headed while hot.

Cast-iron pipe should be inspected for blow-holes, cold-shuts, plugs, or other defects, and tested by hammer for cracks while suspended over the trench; this in addition to testing and inspection at the foundry when made. Each pipe should be sent " home " in the bell. Yarn should be firmly calked into the lead space, of such thickness as to bring the bores of the two pipes into line, and leaving the required depth of space for the lead. The depth may be ascertained by use of a rule applied at various points around the inside of each bell. The lead for each joint should be run in at one pouring, and no cold-lead plugs used. No inspector can tell whether a joint is being so calked as not to leak; the test must develop this fact. It is a good plan to give all the calkers punches of different designs, and have each mark plainly with his punch each joint he calks. A deduction may then be made from the wages of each for all of his joints which are found to leak on testing.

The trenches for buried conduits should be inspected to see that they are of the specified depth and width, dug to the proper line, of uniform grade on the bottom, and that no rock or large stones protrude in the bottom.

Bell-holes for cast-iron pipe should be of such size and location as to permit the barrel of the pipe to rest on the ground; and cross-trenches for riveters on wrought-iron or steel pipe should be of ample size to permit of good work. In back-filling, no large stones should be used before the filling is 2 feet high above the pipe; and selected material should be carefully tamped under, around, and on top of all pipe, especially riveted and wood-stave.

Before the excavation of trenches in open country, the center lines of these should be marked with stakes spaced 100 feet apart on tangents, but closer on curves; and a stake should be driven at one side and directly opposite the point where any special device or structure is to be placed, and plainly marked to indicate what this is—as A.V. for air-valve, V. for valve, H. for hydrant, etc.

In distribution systems the contractor may be given a list

showing the distance of the pipe line from the fence line on each street. But if there be no fence, or the location be by offset from the street center, center stakes for the trench should be set. These should be a uniform distance—say 100 feet—apart, to facilitate finding them; and instead of stakes, large spikes may be used, driven flush with the street surface. A stake should be driven where each fire hydrant is to go, and appropriately marked. Stakes should also be driven, just inside the curbing, opposite the location of each valve, special casting, or other break in the continuity of the pipe, and marked V, T,

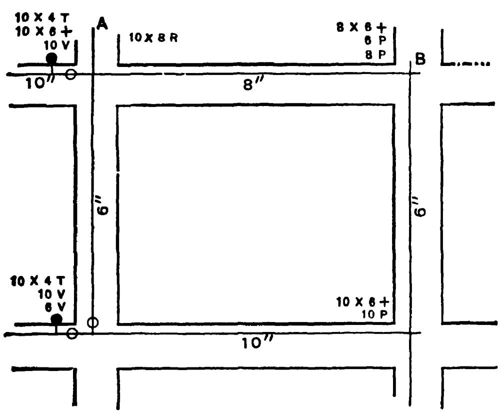


FIG. 110.—Contractor's Pipe and Special Map.

+, etc. After the pipe is strung in the trench, and before calking, the inspector should see that the instructions implied by these stakes have been followed.

The contractor should be supplied with a map of the piping, showing not only the size of pipe on each street, but also every special piece which is to be placed at each point; the latter information to assist the teamster in delivering these where they are needed. Fig. 110 shows a small section of such a map. Thus, at *A* is required a 10×4-inch tee, a 10-inch valve, a 10×6-inch cross, and a 10×8-inch reducer; while at *B* an 8×6-inch cross, a 6-inch plug, and an 8-inch plug are required.

The inspector should see that the pipe is of the required size,

class, and weight (the weight of each pipe is marked inside of one end by the manufacturer) and without defect, and is placed at the correct location and depth; that all specials are placed where directed; that traffic and pedestrians are not interfered with more than is necessary; that blasting and other operations are so conducted as to endanger no lives; that valves and fire hydrants are in good order, stuffing-boxes in shape for service, drip connection (if any) carried to the drain or sewer, hydrants at the right depth, vertical, and well supported and braced as required. He should also see that the back filling and paving are done as specified.

The engineer should keep a note-book in which are entered each line of pipe, its size, distance from the fence or street center, and depth below the surface; with the location of each valve, hydrant, or special of any kind. The exact length of pipe, hydrant connections, etc., is here recorded; the date of beginning and completing the construction, and any special work calling for extra payment.

Bench excavation is generally paid for by the cubic yard of earth and of rock. Conduits of whatever kind are paid for by the lineal foot of each size, weight, band spacing, or thickness of shell; separate payments being made for material and construction, or one payment to cover both. Valves, hydrants, and other special devices are paid for at a price for each in position ready for use. It is customary to measure each line of pipe from end to end, making no deductions for any intermediate specials, these thus being actually paid for twice. Where a pipe changes size by a reducer, each size is considered as terminating in the middle of the reducer. Hydrant branches may be measured as extending from the center of the main pipe to that of the hydrant barrel.

The price bid for laying pipe generally includes excavating, placing and calking the pipe and specials, and back-filling; but rock excavation is generally paid for extra, by the cubic yard. Removing and replacing pavement is paid for as a separate item in some cities, in others is included in the price for laying. The former is preferable.

The price of cast-iron pipe and specials is usually stated per ton of 2000 pounds.

It is extremely desirable to test all conduits and appurtenances under water pressure before they are covered. In sandy soil it is almost impossible to discover leaks in an underground pipe; although in clay or loam these ordinarily make themselves apparent at the surface when at all considerable, unless they find an outlet into a sewer or other underground channel.

If iron pipe is exposed to the sun for several days, expanding by the heat and contracting at night, the joints are made to leak by the motion. It is hence desirable to test each section and cover it the day it is laid. If the section ends at a valve, the end is closed by this for the test; but if not, a temporary or false head may be fastened in the end of the last pipe. A convenient appliance for this is shown in Fig. 111.

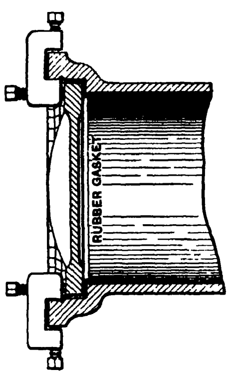


FIG. 111.—False Head for Testing Pipe Line.

If the pumps or reservoir be completed and ready for service before the conduit and distribution system are laid, water for testing the latter may be introduced into them, and the pressure contributed, by these. In many cases, however, the conduits and the reservoir or pump are being constructed simultaneously, and this is impossible. The test may then be made by filling the pipes by gravity or by a small pump from some stream, and applying pressure by a small steam or hand pump. The water can be introduced through a fire hydrant or a special casting left unplugged, or through a pipe tapped into the false head.

The presence of air in a pipe line during a test may result in unnecessary fracture of the pipe. If the advance end of the pipe is higher than the remainder, an opening should be provided here for permitting the air to escape; all hydrants and air escapes should be opened; and all air removed from the

system before the pressure is applied. If a small hand pump is used to raise the pressure, its pipe may be connected with a hydrant nozzle by means of a bushing, or it may be connected with the false head. While the pressure is on, all joints should be examined, and any leaks found should be calked until the pipe is tight throughout and remains so for a specified period; after which the pipe should be covered. Valves and hydrants as well as pipe should be tested under pressure.

While pipe-laying is progressing, the open end of the pipe should be closed with a plug each night to prevent the entrance of animals, or of stones, sticks, etc. Some refuse is almost sure to enter the pipes, however, and when they are first put into service all blow-offs and fire hydrants should be opened and the pipes thoroughly cleaned.

Fire hydrants should be inspected to see that their valves seat properly and are tight, and that the drip acts whenever the hydrant is closed. If the drip does not open, the barrel remains full of water, which is apt to freeze in winter; if it leaks, water is wasted and, if the drip is not connected with the sewer or drain, fills the ground around the hydrant, preventing the latter from draining when shut off.

Art. 85. Other Features

Pumps and boilers are generally set and connected up ready for service by the manufacturer, who is paid a lump sum. In a few instances it has been provided that this sum be diminished at a fixed rate if the plant fall short of a given duty, or increased if it exceed it. The testing of pumping-engines has been discussed in Art. 61.

The foundations of the pumps and boilers are frequently constructed as part of the pumping station. Plans, with dimensions, for these are furnished by the manufacturer, and must be closely adhered to. If bolts are to be built in the foundation by which to tie down the engine, a templet, with holes bored in the proper places through which the upper ends of the bolts pass, should be used for setting them.

In building the chimney, reliance should not be placed solely upon a batter board, but a bolt should be fastened in the foundation at the center of the shaft and plumbed down to after every few feet of added height. The foundation should be perfectly solid, carried down to rock or hardpan if possible, and of such size, unless on rock, as to give a pressure of but $1\frac{1}{2}$ to 3 tons per square foot. It should be carried as deep as the pump pit, or as any other excavation within 50 feet of it, unless it first reach bed-rock.

The above requirements hold for standpipe foundations also. The standpipe is always erected by the manufacturer, and the tests are those conducted at the factory on the material, and that of filling the standpipe with water to determine leakage. A standpipe usually leaks slightly when first filled, but small pin-holes generally fill up in a few days. After the pipe is erected, but before painting, it should be carefully examined for riveting cracks or other flaws. Defective plates can be removed from any part of a completed standpipe and replaced.

Wells are driven sometimes by the city or water company, more often by contract at a fixed price per foot. In some instances the contractor guarantees a certain minimum yield; in which case the guarantee should extend through six months or more of actual service, and should designate the maximum depth to which the ground water will be lowered at the well during pumping. In making a test measurement, the water discharged should not be permitted to flow upon the surface near the wells, as it may then enter the ground and be repumped. Samples should be taken every few feet of the material through which the well passes. Care should be taken to have every joint of the casing made tight.

Purification plants of the slow sand type are generally in the form of structures of concrete containing an underdrain system and sand. The concrete is constructed as for reservoir work, with special care to secure water-tightness at the bottom and strength in the roof. The sand should be closely inspected to see that it is clean and of uniform size within the limits specified. It should be deposited in such a way as to secure

uniformity of packing—no walking or working on it should be allowed.

Rapid sand filters are more complicated combinations of concrete, sand, piping, and various controlling devices. Except for occasional large plants, they are let as a whole to parties making a business of constructing them, under specifications defining the area, and, in a general way, the amount and nature of automatic and manual controlling applicances, apparatus for preparing and applying chemicals, whether a clear water basin is to be included (an existing reservoir can sometimes be used for this purpose); also a guarantee by the contractor of the maximum bacterial content of the effluent.

Liquid chlorine apparatus is purchased entire and set up in a small area of an existing building or in a small one (8 or 10 feet square is ample) constructed for the purpose.

All designing of filtration, softening or other special treatment plants should be entrusted to those who have made a specialty of this, and it is generally desirable that the construction also and the operation during the first few weeks be supervised by them.

CHAPTER XVI

PRACTICAL CONSTRUCTION

Art. 86. Reservoirs

Probably the easiest way to clear wooded land is to dig down to and cut off all tree-roots a foot or more beneath the surface, and then pull down the trees by a rope previously attached to them some distance above the ground. There is then no stump to remove. If there are stumps, these may be burned out, pulled out by a stump-puller, or blown up by dynamite. The trees being down, the brush is next cut with sharp brush-hooks (the Spanish-American machete is excellent for the purpose.) This and the tree-tops may be burned in piles. A contractors' plow may then be used to loosen the soil for a foot in depth; a heavy-framed, long-toothed harrow will drag out most of the larger roots; and the smaller roots and loam can then be removed by scraper. This top loam should not be used for the body of embankments, but may be used for a top dressing on the outside slopes. Land to be flooded should be cleared of all vegetable growth just before such flooding takes place, whether or not it has been cleared before.

For dressing up and banking the shores of the reservoir, casting by shovel and short wheelbarrow runs will generally be found the most effective methods. Where the soil is of loam, sand, or gravel, or any material easily loosened by plow, the scraper is probably the most economical method of excavating and embanking; unless the haul exceed about 1000 feet, when a cart, filled by hand or by steam shovel, is more economical. In either case, except where a steam shovel is used, the ground should be well loosened by means of a rooter-plow and a 2- to 6-horse team. A wheel-scraper is preferable to a drag for most work. In making embankment several scrapers should

be used, and should travel a regular route, coming on the embankment generally at one or both ends, and leaving it at the other end or the middle. One scraper to 300 or 400 feet of distance traveled by them will generally keep all hands busy, there being one man in the hole loading scrapers, one on the bank to dump them, and a driver accompanying each team.

If carts be used, filled by hand, two are generally provided for each team, one being loaded while the other is being hauled to the embankment and dumped, the number of men loading each cart being just sufficient to fill it while the team is absent with the other. This number will of course depend upon the length of haul and ease of shoveling. If the haul is of considerable length, it will be better to use three carts to two teams, or even four to three teams. Under some conditions, where time is a very important consideration, teams may be distributed at a uniform distance of 50 to 100 feet apart over the whole course and pass continuously around the circuit, being loaded by receiving a shovelful from each of 50 to 75 men distributed along the route as they pass them, and stopping only to be dumped. But this is not generally an economical method. Where there is a sufficient force of carts, embankment spreaders, rollers, etc., to keep it busy, a steam shovel is more economical than hand shoveling.

It pays to keep in good shape the road by which carts or scrapers travel from the pit to the dump. On a clay or sandy soil, which is heavy in wet or dry weather respectively, a good road can be made of plank spiked to two 12-foot stringers and made up in sections 8 feet long, with about 2 feet of each stringer protruding beyond each end of the section; alternate sections having their stringers at the ends of the road-plank and about 1 foot from the ends, so as to lap when the sections are placed together. This forms a portable road which will give a good surface if laid on leveled-off ground. The width may be 6 or 8 feet; and 18 or 22 feet B.M. per running foot is required for these widths (see Fig. 112).

Wetting of the embankment should be done by a spraying nozzle on a hose or water cart. Puddles are formed if a bucket

be used. In case a stream is flowing near by above the elevation of the embankment, this can be brought for use by constructing a temporary earth, timber, or brush dam across it and from this carrying a flume to a point above the embankment. Here a gate can permit of filling a sprinkling-cart, or a hose can be attached, the surplus water discharging from the flume below the embankment. This method can often be employed on impounding reservoirs. If the flume would be too long, however, or the temporary dam too expensive, it may be cheaper to fill the sprinkling-cart directly from the stream by means of a boat pump, diaphragm or other small pump, and cart it to the embankment. In constructing a distributing reservoir, carting water is generally necessary.

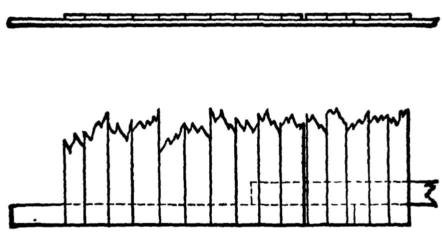


FIG. 112.—Portable Plank Road.

If a horse-drawn roller be used, the best kind is one with grooves at least 2 inches deep and wide in its face. When a steam roller is used, the wheels may be smooth faced, but the author prefers grooves for this also.

For dressing the face of an embankment, a broad grub-axe is the best tool. A foothold is furnished by a plank having cleats nailed to it about a foot apart, which is kept from sliding down the bank by a stake driven at its lower end. The bank is first graded under the templet, beginning at the top, and is surfaced by eye between templets. Selection should be made by trial of men who can obtain the most uniform surface in this manner. The surplus dirt removed in this dressing can be used for filling in low places left for this purpose, or on the outside of the embankment.

Puddle, when made by mixing different materials, should

be measured and turned over carefully by hand or in a mixer with as much thoroughness as is concrete, before being put in place. It may then be thoroughly rolled; but a better puddle is made if it be rammed by barefooted or rubber-booted men with small-headed rammers, or if trodden by cattle. In India, goats are much used for this purpose, and they were used on the Santa Fe dam in this country. If there be lumps of clay in the material, these should be fine-cut by spades before mixing; but if ready-mixed hardpan be used, this may first be rolled to break up the lumps, and then wetted and puddled. Puddle should never be wet, but only quite damp, and the materials should be thoroughly mixed.

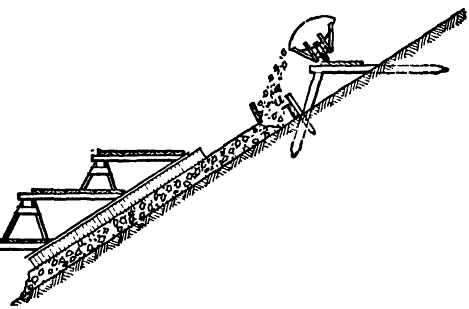


FIG. 113.—Concreting and Lining Slopes.

Broken stone for lining is generally carried to the top of the slope and from there lowered to where wanted, wooden chutes being convenient for this purpose. Slope-wall stone is delivered to the workmen in the same way. But derricks are to be preferred if the work is of considerable size.

Concrete for slopes is sometimes delivered through chutes directly from a mixer on the top of the bank; but when it is, and is as dry as is generally necessary, it must often be remixed before using, as the stone is apt to separate out. It is better, on small jobs, to construct a wheelbarrow run along the slope, maintaining it at such a position, by frequently moving it up the slope, that the material can be dumped directly upon the top of concrete already in place. It should never be dumped upon the ground, to slide or roll down the slope. If a plank

be held as shown in Fig. 113 to catch the concrete when dumped, none need be wasted. The run can rest upon supports driven in the bank as shown, and spaced about 6 feet apart.

On large jobs the use of tower and chutes is common; but this method of handling concrete requires that it be thinner than can be handled on slopes; besides which, it is believed that a concrete is more impervious if less water be used than this method requires.

Concrete on slopes is generally laid in horizontal and not vertical strips; as by the latter method that at the bottom, not being set before the top is placed, will be pushed out of position by the weight of that above, and the whole strip of concrete tend to slide down the bank. Concrete lining should be lightly rammed, and carefully brought to surface.

Concrete for the bottom should be deposited where needed, and never shoveled up off the ground. A paving concrete mixer can be used, depositing the concrete directly in place, the cement and aggregate being brought to it by wheelbarrows over bottom not yet concreted. Both here and on the slopes, the proper thickness and surface elevation can be obtained by the use of steel or wooden forms, such as are used in concrete pavement construction, set accurately to grade.

If a masonry dam be less than 20 feet high, probably the best way of handling the stone is by derricks on the ground. If it be higher than this, derricks can be placed upon the dam, and provided with main-falls sufficiently long to reach the ground and raise the materials from there. But the derricks must be raised continually as the dam rises; and this raising is a source of expense, and of delay unless done out of working-hours.* To permit of raising, the guys should be adjustable in length. A gin-pole can be used for lifting the derrick-masts.

In some instances a traveling derrick is used, the track on which it runs being supported on a scaffold which rests against one face of the dam, or on a timber trestle paralleling the dam. The cableway has been much used for bringing material to the

* In constructing the New Croton dam, the derricks were mounted on steel towers, which were built into the masonry.

Fig. 114.—Use of Derricks and Cableways in Building Ashokan Dam.

Stiff-legged derricks used in setting stones that are brought to them by the cableway In the background, behind the cableway towers, is seen a core wall for earthen embankment.

dam, and is generally the most economical contrivance when the dam is of considerable size. Spans of 2000 feet and more have been used. The cable is generally used for bringing the material from the cars or carts at one end of the dam to where it is needed but not for setting the stones, both because the cable extends only over the center line of the dam, and because its swaying prevents a nice adjustment of the stone. Derricks are generally used for setting the stone.

Water for mixing mortar and washing the stones can be brought by flume, as for embankments, or in casks by derrick or cableway. Mortar should not be mixed on the wall, for several reasons.

The faces of small concrete dams may be made against forms; but for large dams the more common practice is to build the faces of stone or concrete blocks, the space between being filled with concrete thoroughly worked into complete contact with the backs of the blocks. The blocks are kept about one course above the concrete. Even for small dams this gives a better appearance than when forms are used.

In the construction of impounding reservoirs, the handling of the stream is frequently troublesome and difficult. It is generally necessary to carry it over the heart-wall trench in a flume. The outlet pipes and base of the gatehouse may sometimes be constructed first, and the stream turned through these; it being meantime carried around the structure in its natural channel, or in an artificial canal or flume, into which it is turned by a temporary diverting dam. The great danger is from sudden floods; and until such time as the dam forms a basin capable of holding so much of the entire maximum yield of a storm—say 3 to 5 inches over the entire drainage area—as the outlet pipes will not discharge, this dam and flume should be able to carry such flood around the works. Should water overflow an embankment it may destroy it. If a masonry dam under construction is overflowed, it may afterward be necessary to remove the top stones because of cement washed out of and dirt deposited in the joints. Derricks and all movable plant are likely to be washed away, also. It is of course desirable

to build the lower part of a dam during a season free from rainstorms; and this is particularly true of a cut-off wall, or such part of the dam as is below the natural surface.

ART. 87. DISTRIBUTION SYSTEM

Iron pipe is generally delivered to a contractor in gondola or flat cars on a siding, and must be unloaded and distributed by him. Ordinary 4-, 6-, and even 8-inch pipe can be loaded onto a wagon by rolling it down two skids about 9 feet apart placed from the wagon to the top of the car-side; it being controlled by a rope passing around each end, one end of which is fastened to the car and the other held by hand and payed out. Two men in the car lift the pipe onto the skids and hold the ropes, and one in the wagon receives it. For heavier pipe, three or four men in the car and two in the wagon are required; or a yard-derrick or portable stiff-leg may be used for unloading.

The pipes are distributed along one side of the street, and care should be taken that they are the right size and class, and that just enough are strung along each block to lay the line without any rehandling. In unloading small pipe, one end of the pipe is placed on the ground while the other rests in the wagon; a bag of hay or a woven rope pillow is placed under the tail of the wagon, upon which the pipe fails when the wagon is started forward. Large pipe should be unloaded by skids, as when unloading cars, the ropes around the pipe being payed out by winches operated by hand, or by the engine of the motor truck if such a truck be used for hauling the pipe. Pipes are generally delivered on the side of the street opposite that on which they are to be laid.

In excavating trenches, the paving is generally placed on the side toward the center of the street, thus serving as a guard for vehicles, and the dirt is thrown toward the sidewalk. By this arrangement the pipes do not need to be lifted over the dirt-pile when placed in the trench. Fig. 115 shows a method of preventing the excavated dirt from blocking the sidewalk and from stopping the gutter-flow. The first earth cast out should

be thrown to what will be the outside edge of the bank, since it cannot be thrown there when the trench is deeper without double handling.

Excavating in winter is expensive, but must sometimes be done. It is facilitated by thawing the surface along the line of the trench. This has been done by the application of manure for a day or two before excavation, or of salt. A quicker method is to use boxes without ends about 24 inches wide and 12 inches high, placed in contact end to end on the line of the proposed trench, all joints and the ends being covered with bagging and earth. These are then filled with steam from a portable boiler through a steam hose. The cost of thawing by this method in Providence, R. I., was 1.7 cents per square foot.

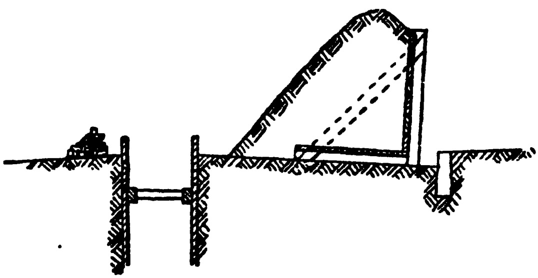


FIG. 115.—Excavation Platform.

In blasting rock, the drill-holes should be 3 or 4 inches deeper than the required trench bottom; with certain kinds of rock or pitch of strata still deeper holes may be necessary. Each blast should be covered by mats of woven rope or bundles of branches or small trees tied into fascines, or by planks or logs chained together; the former better preventing small stones from flying. If the earth banks near by are not quite solid, they should be braced before the blasting. In case of boulders it is often cheaper to remove these bodily than to blast them in the trench at risk of caving the banks.

Trenching machines are used in favorable soil, these being in the form of a revolving wheel carrying scoop buckets on its circumference, or a double link belt carrying numerous similar buckets and carried by an adjustable boom. Or a steam shovel

with a long dipper arm is used. These are troublesome to operate where numerous service pipes cross the trench.

It is desirable to place the pipe in the trench as soon as possible. If caving of the banks occurs afterwards, only the bells need to be uncovered for making the joints.

Four-, 6-, and 8-inch pipe can be lowered into the trench by two men, by means of ropes passed around either end of the pipe. Each man slips under one end of the pipe, as it rests on the bank of the trench, a knotted end of a rope. Standing on this he holds the other end in his hand, and, by paying it out, lowers the pipe into the trench. Then, straddling the bank, these men lift the pipe clear of the trench bottom, while a third pushes it home into the bell of the pipe last laid, a fourth meantime passing a strand of packing around the spigot of the pipe just back of the bead, and guiding it into the bell. The third man is provided with a wooden bar which he thrusts into the bell-end a foot or so, and by which he pushes and lifts the pipe. With this, under the foreman's direction, he shifts the pipe into the correct position. A straight line of pipe gives joints that are more easily made and stronger than does a crooked one, it has a more workmanlike appearance, and causes less friction of water passing through it.

For heavy pipe some other method of lowering is necessary, and some form of derrick is generally used, one light and easily portable being necessary. Fig. 116 shows a good appliance for this work, for heavy pipe. The derrick is carried along the trench by three men, and is rolled from street to street on its own wheels. For light pipe the drum and wheels may be omitted, the fall being payed out by hand. While being lowered, the pipe is guided by the two men in the trench. Before lowering, the pipe is rolled onto skids which span the trench, and which are removed when the pipe is suspended from the derrick. Some contractors have used a portable derrick for lowering heavy pipe, others the dipper arm of a steam shovel, and others the heavy traveler of a sewer excavating machine.

The laying of pipe should be begun at a special whose position is fixed—as a valve, or an intersecting pipe-line. The

pipes are laid in succession until near where another special comes, when this special is put in place and the closing length of pipe—which will generally be less than a full length—is temporarily omitted, the laying beginning again with this special. Behind this gang come two men who fit and cut the closure. The length required is accurately measured, and a piece about

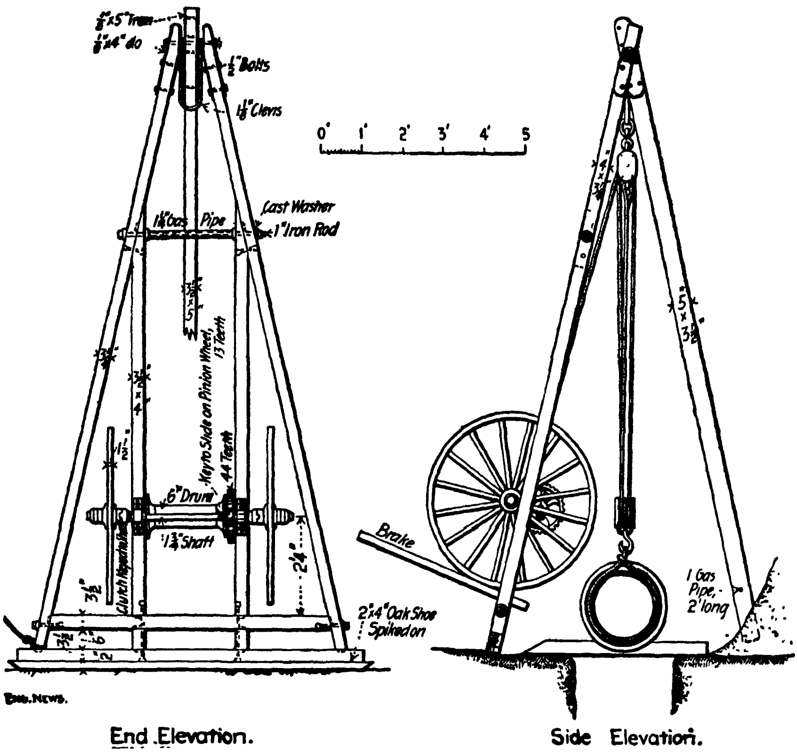


FIG. 116.—Pipe Derrick.

$\frac{1}{4}$ inch shorter is cut from a pipe, and is sprung into place by raising one or two pipes on either side of it, entering spigots into bells and dropping all into position again. In cutting cast-iron pipe, a ring is marked around the pipe for a guide, a 4 by 4 to 6 by 6 pillow-block is placed under the pipe at this ring, and by successive applications of a dog-chisel (Fig. 117, 1) and an 8- or 10-pound sledge a slight cut is made around the pipe.

This cut is made deeper and deeper, the pipe being meantime rolled back and forth along the pillow-block, and the blows of the sledge being made heavier and heavier, until at length the pipe breaks off at this ring. If the iron be good and the sledge-blows not too heavy at the beginning, there is little danger of breaking the pipe except where desired. Iron chips are apt to fly into the eyes of the workmen during this operation, and eye-shields of transparent celluloid are recommended for use.

Fire hydrants are set and their branches laid, generally by this second gang, and the blocking placed behind the hydrant.

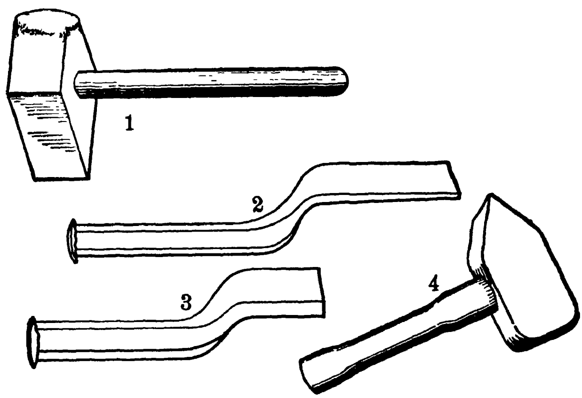


Fig. 117.—Yarning and Calking Tools.
1. Pipe-cutting chisel. 2. Yarning iron. 3. Calking tool. 4. Calking hammer.

All valves and specials and other appurtenances requiring lead joints are set in position, in order that the lead-gang may not need to revisit this line.

After the pipe is laid a "yarner" places the packing. A tightly twisted length of packing, long enough to go around the pipe, is inserted between the bottom of a bell and its spigot, and pushed back to the shoulder of the bell; the spigot being lifted, if necessary, by driving a steel wedge between it and the bell, until the annular space at the bottom is slightly greater than that at the top. The packing is then calked tightly back against the shoulder of the bell all around, more being added where necessary to leave just the required depth of lead space. Tarred packing can be calked harder and is easier to handle

than untarred. To hold the molten lead in the joint when it is poured, either a "clay roll" or a jointer is required. The former is made of dampened fire-clay molded around a strand of rope a little longer than the circumference of the pipe; but the asbestos jointer has almost entirely superseded the home-made roll. This is placed around the pipe and pressed firmly against the bell face, the two ends being brought together on top, but leaving a pouring-hole, into which the lead is poured slowly but steadily until it is about to overflow the pouring-hole. In ten or fifteen seconds the lead will be hard, and the jointer may be removed to the next joint.

The lead is melted in a kettle supported in a small portable furnace in which charcoal is used for fuel, or an oil-burning furnace made for this purpose. The molten lead is carried from the kettle to the pipe in a ladle, from which it is poured into the joint. If the pipe be larger than 10 or 12 inches, a two-handled ladle is necessary, handled by two men both on the surface and in the trench; and for very large pipe the lead is poured from a kettle suspended over the joint by a light derrick.

When the jointer is removed, the lip of lead at the pouring-hole is cut off, a chisel is driven lightly between lead and pipe all around, and the calking tools are then used (see Fig. 117). The lead at the bottom is compacted first, and calking proceeds up both sides until the top is reached. In calking pipe up to 16 or 18 inches in diameter, the calker passes one arm around each side of the pipe, holding the calking-tool in one hand and the hammer in the other. For calking the bottoms of joints in very large pipe the men work in pairs; each hooking a leg and arm in that of his fellow, one holds a calking-tool and the other the hammer in his free hand, and they are thus able to reach the bottom of the pipe. Some contractors have used to advantage pneumatic calkers (operating on the principle of pneumatic riveters) operated by portable air compressors that are wheeled along the street near the trench. The lead should be upset and thoroughly driven until there is no more "give" to it. The lips and other pieces of lead are saved and remelted.

The line is now ready to be tested; and after all leaks that are revealed by the test have been stopped by recalking, the trench may be refilled. The backfilling should be thoroughly tamped in 6-inch layers, if in a street; but if in fields or woods, this expense may be avoided.

In placing the ordinary extension valve-boxes, their tops should be set accurately to grade if the trench is to be well rammed; but if the backfilling is thrown in untamped, the top should be set a little below the top of the filling, and perhaps

Fig. 118.—Carrying 42 and 24-inch C. I. Mains across River by Cofferdam Method.

twice as much above the original street-surface; when the trench has settled they will then be flush with the surface, as they should be.

In crossing streams under water the methods described in "Sewerage," Art. 74 (seventh edition) may be employed. Many other plans have been followed, however, the details varying in almost every case. In one case a 72-inch pipe was put together in two sections, each about 95 feet long, bulkheads were built in each end of these, they were floated over a trench dug in the bottom to receive them, and sunk to place by admitting water

into them, the abutting ends being connected by a diver. In another case a main 119 feet long, weighing 63,300 pounds, 48 inches diameter, was lowered as a whole by derricks. In other cases pipes have been lowered singly and jointed by divers. But in the majority of cases flexible joints are used, generally at intervals of several pipes. The pipe is then laid from scows, through the ice, or connected on shore and dragged across the river bottom. Joints can be made under water with ordinary bell-and-spigot pipes by use of lead wool calked into the

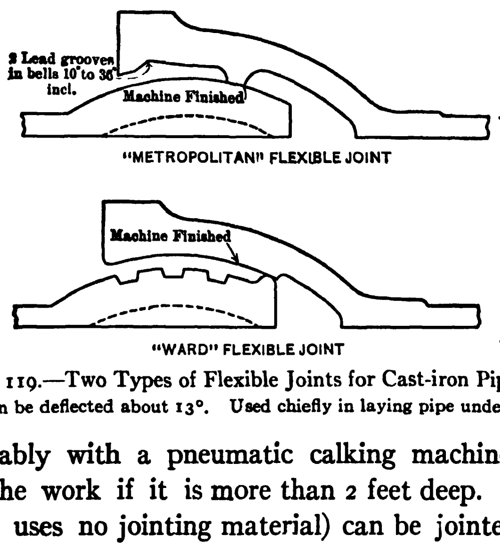


FIG. 119.—Two Types of Flexible Joints for Cast-iron Pipe.
Either can be deflected about 13°. Used chiefly in laying pipe under water.

joint, preferably with a pneumatic calking machine, a diver performing the work if it is more than 2 feet deep. Universal pipe (which uses no jointing material) can be jointed together under water readily and rapidly, and the joints are "flexible" in one direction.

ART. 88. WELLS.

Dug wells for water works are generally not less than 12 feet in diameter, and must always be sheathed and braced. They are generally circular in plan, since a given amount of material will thus make a stronger wall and give greater capacity. The sheathing, however, is seldom made circular, owing to the difficulty of forming the wales or ribs. (Sheathing is not required when the walls of the well are built above the surface and sunk bodily into the ground.) It is generally made square or octagonal in plan, each course of wales being composed

of four or eight lengths of 4 by 4 to 12 by 12 timber, strongly braced together at the ends, as at *a* and *b*, Fig. 120. It is not advisable to make any one wale-piece more than 8 or 10 feet long, and 6 feet is a safer limit unless braces are used across the well. This limits an unbraced well to 12 to 20 feet outside, or say 9 to 16 feet inside, diameter. Braces across a well which is larger than this require to be stiffened against buckling, and the danger of collapse increases rapidly with the diameter. For

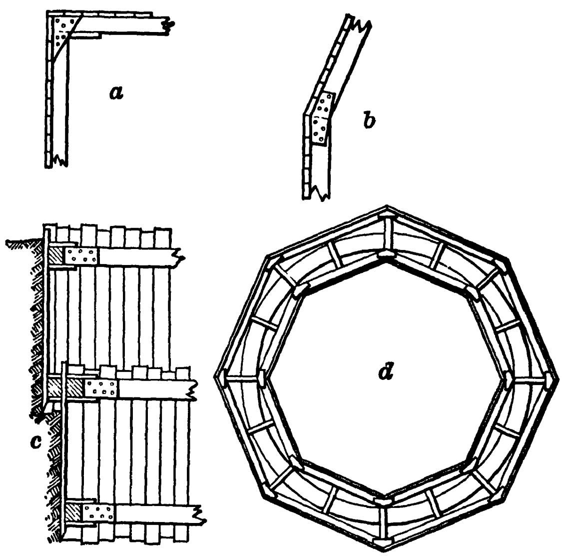


FIG. 120.—Sheathing Dug Wells.

large wells the method *d* may be adopted, the excavation being practically an annular trench around the earth core *d*, which can be well braced, and will not collapse because of the failure of one brace or wale. This trench is carried to the required depth and the wall is built in it, earth being packed between the wall and the outer sheathing as the former is built, and all braces being left in. After the wall is completed the center core is excavated and inside sheathing removed. The braces may be sawed off at the inside face of the wall; or may be pulled out and the openings in the wall filled up, or these may be

left to serve as weep-holes. This construction can be used for any size, shape, and depth of well.

With either of these methods the earth is best removed by "staging" it out, down to about 15 feet, below which a derrick and large dirt-buckets are more economical. The derrick may be placed on *d* in the annular-trench plan, but is preferably placed a few feet back from the well on firm soil. The boom should be long enough to remove the dirt some distance from the well; unless it be immediately carted away, which is the better plan, since the weight of extra dirt on the surface around the well adds to the strain on the wales and braces, and none should be placed within 20 or 30 feet of the excavation. At 30 feet depth a horse-power derrick can handle three half-yard buckets filled by six men; below this not so many diggers per derrick can be used. With a steam-power derrick four 1-yard buckets and 15 to 18 men can be kept busy. These quantities are approximate averages.

When wood sheathing is used, planks longer than 10 or 12 feet cannot ordinarily be used, and for wells deeper than this, double sheathing must be used, as at *c*. But steel sheet piling is in common use now, and this can be driven to much greater depth.

Another method of sinking wells, which does away with bracing, consists of sinking the masonry lining into the ground by its own weight. A stiff timber or iron footing is placed in the bottom of an ordinary circular hand-dug well 8 or 10 feet deep, and on this the masonry wall is built. When the wall has been built up to the ground-level, the soil is excavated from under the footing equally all around, and the wall sinks of its own weight. This operation is continued, the wall being kept built up to the surface level, until the required depth is reached. The footing generally extends several inches outside the wall, and is frequently provided with a sharp cutting-shoe around its outer edge. The outside of the wall should be plastered and made smooth, or board sheathing placed around it nailed to furring built in the wall, to reduce the friction between the wall and the soil if the latter settles against it, as it generally does. The

greatest danger is that the well will get out of plumb by the more rapid settling of one side than of the other. If the wall "hangs" at any time and refuses to settle vertically, the excavation under the footing should be stopped at a uniform level all around and not more than 6 to 12 inches under the footing, and not continued until the wall has settled down to a bearing. Care must be taken that the excavation under the footing is at all times level. A "hung" wall is lowered by building it higher or loading it on top with pig iron, helped in extreme cases by the careful application of water-jets around the outside of the well to loosen the soil.

In all well work the water must of course be kept out. For this purpose a pulsometer is excellent for handling small amounts of water, a centrifugal pump for larger amounts. Reciprocating pumps are sometimes used, but are subject to great wear from muddy water. A centrifugal pump is frequently run by belt from the surface. If electricity is available, an electric motor geared to the centrifugal pump is advisable, since the belts are apt to slip in a damp well, and are greatly in the way.

Pump pits are built in the same manner as are wells, except that their walls are made heavier and water-tight and a water-tight foundation is provided. Concrete is almost always used for pump pits, and often for wells.

Small 1½- or 2-inch wells not more than 30 or 40 feet deep are sunk through the softer soils by simply driving them down with a hammer; but this is generally impracticable through hardpan and large gravel, and of course through rock. Such small, shallow wells are not advised for water works, but 4- to 12-inch wells through clay or rock should generally be employed. The best method of sinking these in most cases is by the "jetting down" process. A small pipe, whose diameter is about one-third to one-half that of the well-casing, is placed inside the casing and water is forced through it by a pump, passing up between the two pipes and out at the top of the larger. The jet from the bottom of the water pipe loosens the soil, and it is carried up by the ascending water. As the soil is thus re-

moved the casing is driven down by a hammer. The water-pipe is generally given a churning up-and-down motion in firm soils. In sand or loam it will in most cases drop more rapidly than the casing can be made to follow. An arrangement for this work is shown in Fig. 121. As both water and casing pipe are lowered, they are extended by screwing additional pieces onto

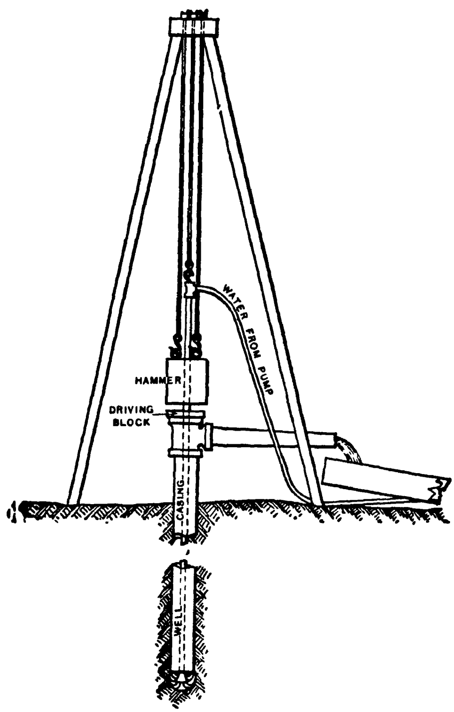


FIG. 121.—Apparatus for Jetting Down Wells.

their top ends. The casing is held upright, and the water pipe is supported and churned, by a derrick. Ropes from the water pipe and hammer pass through sheaves at the top of the derrick to a two-drum hoisting-engine, or a cam-and-lever well-boring engine. In sand and clay 10 to 20 feet per hour can be sunk by this method. The most difficult material is gravel, the jet-water passing out through the gravel and not up the casing.

Rock of course cannot be bored by this method, but when this is encountered either a diamond-drill is used, or, as is more common, a two-, three-, or four-sided cutting-drill, to which are fastened drill-rods and "jars" to increase the force of the blow, the drill being raised, revolved a few degrees, and let fall with each blow. The drill is lowered through the casing. If the casing is to follow through the rock, an expansion drill is used. The chips are washed out of the hole by water-jet.

A casing can generally be driven 400 to 800 feet, after which the friction becomes so great as to prevent further motion. A smaller casing is then lowered inside this and carried down another 400 to 800 feet.

The casing is generally lap-welded wrought-iron pipe, with screw-joints; the coupling being made by a sleeve, or by a "flush-joint," i.e., half the thickness of the metal is removed from the outside of one pipe and the inside of the other for a distance of 3 to 6 inches from the end, and threads are cut on these, the pipes when screwed together being thus flush both inside and out; or by an "inserted" joint—one end of each pipe being expanded and threaded inside, thus practically forming a sleeve. In the California or "stovepipe" well, two sizes of pipe are used, one just slipping inside the other, each of them of No. 10 to 14 gauge steel; the lengths being so placed that the inner and outer pipes break joints. This gives a smooth surface both inside and out.

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